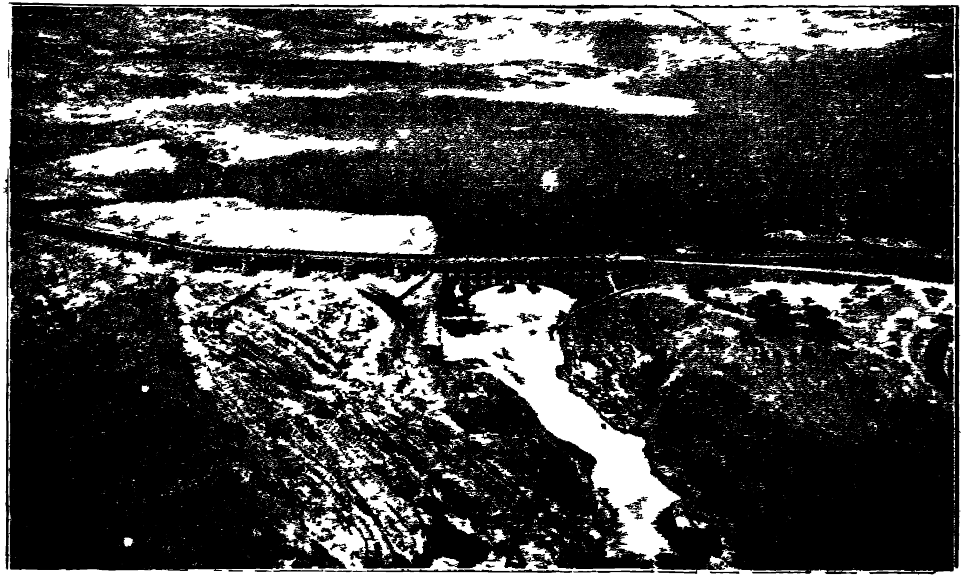
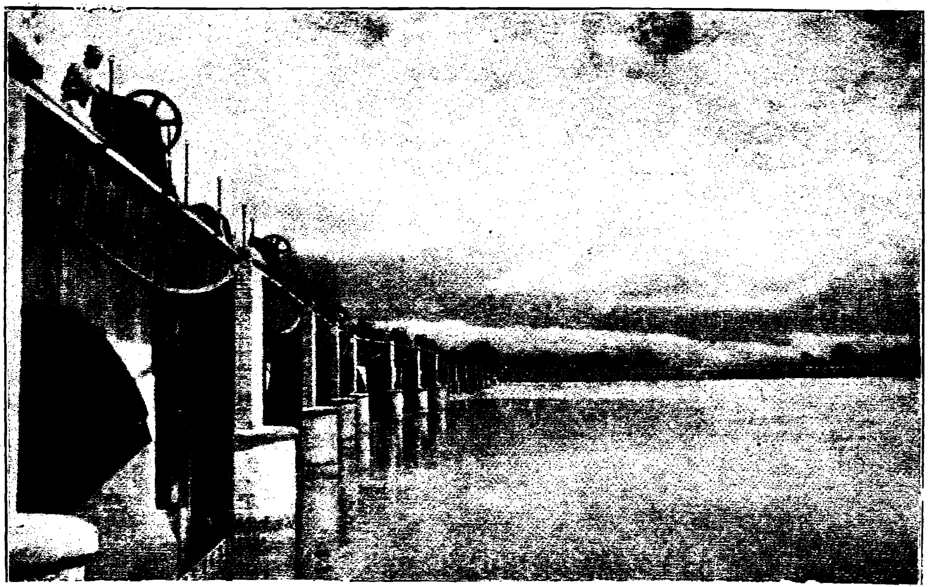
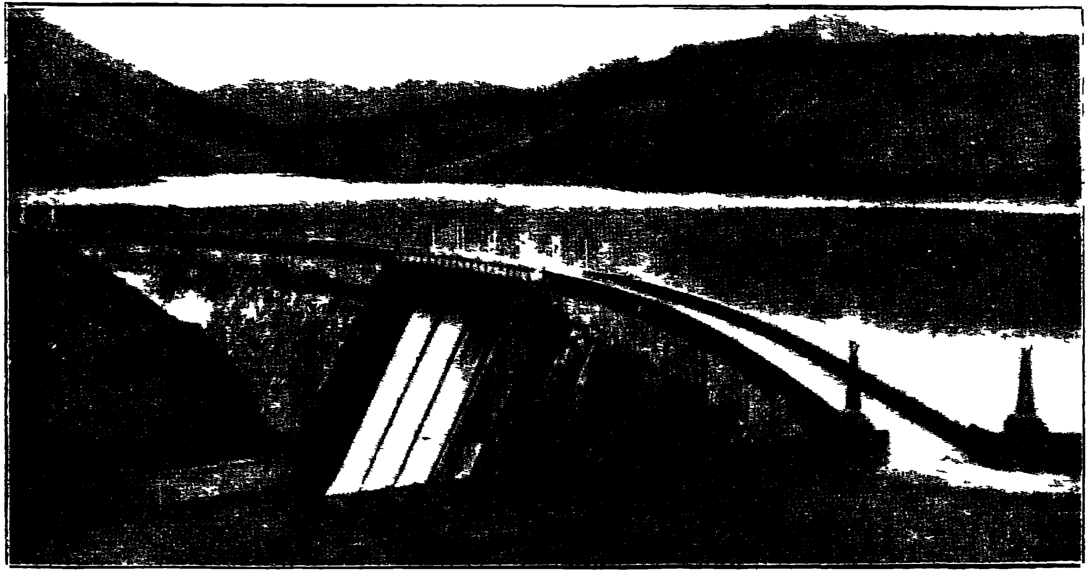


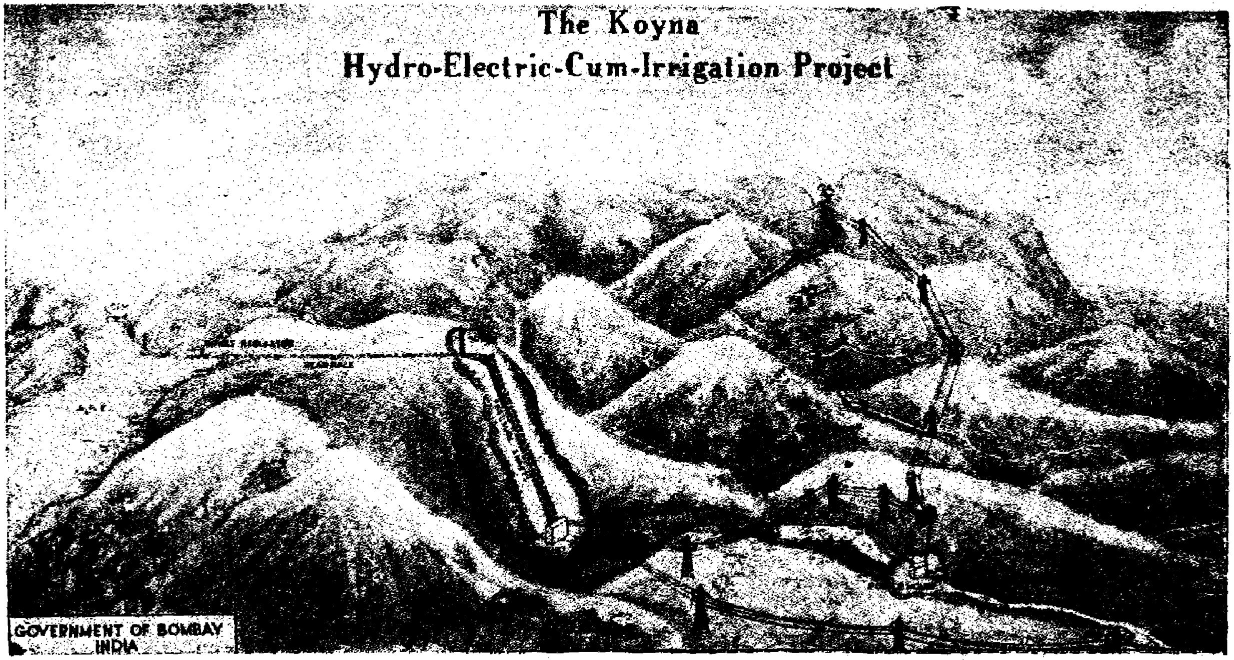
The Krishnarajasagara dam near Mysore.



The Grand Anicut, Madras Presidency.



The Kundale Dam, Travancore-Cochin-State.



Intake Regulator — Surge — Approach Tunnel

CHAPTER I

INTRODUCTION

1. Irrigation is defined as the artificial application of water to soil, for the purpose of supplying water essential to plant growth. It is a means by which water is conveyed to arid areas from rivers, reservoirs or wells to increase the fertility of the land. Where rivers are the sources of water supply, weirs (or on a large scale barrages) are used to raise the level of the water to that of the irrigation canals. In many instances, instead of weirs, reservoirs are built in order to conserve the supply and also to regulate the flood waters of the river.

Scientific irrigation involves a knowledge of the available water supply, its conservation and application to the land, the characteristics and needs of the different types of soil, and the requirements of the various crops to be produced. It is the science of harnessing the sources of water and distributing the same for agriculture.

2. Necessity for Irrigation : When the rainfall in a locality is either insufficient or is not got in time (when required), irrigation becomes a necessity.

A crop requires a certain amount of water to begin with and then it requires some quantity, at intervals, for its growth, until the crop matures. If the rain can satisfy both the above conditions, no separate irrigation is required. For instance, in England the rainfall is sufficient and distributed throughout the year, which is enough for the crop to mature, hence there is no necessity for irrigation in that country.

Similarly in India, we have such typical areas, for example narrow belt of land to the south of the Himalayas, belonging to Bengal and Bihar; also the western portion of the Western Ghats (a typical area). In these places no irrigation is necessary.

But in tropical and sub-tropical countries, the above mentioned two conditions do not exist, and consequently the necessity for irrigation arises.

The conditions where the necessity for irrigation arises has been summarised as follows :

(1) When the total rainfall during the year is not sufficient
: the crop, that is, if the rainfall is less than 40 inches, the crop
nnot be grown without irrigation of some magnitude. Artificial
igation is therefore necessary to mature the crop.

(2) The rainfall for the year may be sufficient, but it may
ot be distributed throughout the crop period for the benefit of the
op. For example, Madras State gets a rainfall of 40–60 inches
nd near the slopes of the Himalayas the rainfall is 50–100 inches.
Even here, artificial irrigation has been found necessary, because
the rainfall is not distributed throughout the crop period.

(Sometimes artificial irrigation is practised to grow commercial
crops or to get a better yield from the lands).

The water for irrigation is taken directly from a river, or
stored in a reservoir. Take for example,

(a) Sind, the Punjab or Egypt. Here the rainfall is small,
being only from 3 to 5 in.; and so the rivers in the locality have
been harnessed to supply the required water.

(b) Northern India. Here no storage of water is required, because the rivers in the locality are all perennial, coming from the Himalayas, and the canals are taken directly from the rivers.

(c) In Mysore, the Deccan and Gujerat, the streams are practically dry in the hot weather and there is not enough rain. Hence reservoirs are built here to store water when it rains and water required for irrigation is taken from them.

In general, it may be said that irrigation is most extensively
ractised in arid regions where agriculture, without it, is precarious
impracticable; and in semi-arid regions, it is used to increase
e general yield of ordinary crops and for special crops like paddy,
ugarcane, vegetables, etc.

3. Importance of Irrigation Works: For a country like India with its small or fluctuating rainfall, the importance of irrigation cannot be overlooked, and topmost priority should always be given to it. Other works, like railways, are no doubt beneficial to a country. They level off rates and costs, and cause a uniform
... the whole country, though there are good many advan-
... whereas irrigation is an actual producer of food and
... ntry from famine and distress. Also even if the
... the grain from the locality and raises the rates,

still the sale proceeds of the crop remain only with the cultivators and compensate them.

4. Functions of Irrigation : (*a*) The most important function of any irrigation work is the ***protection of the area against famine.*** For this purpose, the works concerned should be executed with proper care regarding the design and construction, special attention being paid to the quantity of water-supply available permanently at the locality.

(*b*) The other function of irrigation is the improvement in the kind of crop grown in the area; that is ***growing a superior crop in the place of an inferior one.*** This is achieved through the permanent supply of water, where the tract was dependent formerly on fluctuating rainfall. For example, wheat generally replaces barley, and sugarcane and indigo replace the millet crop.

(*c*) As a result of the assured water-supply, the lands become more fertile and attract people to the locality. For example, many ***desolate and dry places, which were very scantly populated, have become populous, after the development of irrigation in that area.***

5. Development of Irrigation : To develop irrigation in any country, the following measures should be adopted :

(1) The people should be trained to become irrigation minded.

(2) The land should be fertile or made fertile within reasonable cost.

(3) The rainfall should be sufficient and well distributed throughout the crop season, or the water required for the crop should be at all times made available until the crop matures.

(4) Remunerative crops such as onions, potatoes, sugarcane, etc. should be investigated and the people induced to grow them.

6. Natural Facilities for Irrigation in India : India is a vast country (a sub-continent as it were) with a climate varying from extreme cold to the hottest, and with a rainfall of practically no rain or 3 in. in the Rajaputana desert to 326 in. in Chirapunji in Assam. It consists of plains, plateau, high and low mountains and valleys. Thus the irrigation in the country has developed according to the local conditions in each part but everywhere, practically, there is good facility for irrigation.

What is required to achieve this end is (1) land, (2) and of course (3) people. India possesses all these requirements.

(1) **Land**: There is plenty. It is one of the bi countries in the world having an area of 1·5 million sq. mil even, say, possessing 1000 mil. acres of cultivable land.

(2) **Water**: Recent research work has shown that discharges into the sea a surplus of 20 lacs cusecs of water. quantity is gauged in its rivers alone. In addition to the s the country possesses a huge quantity of subsoil water or ground water and of this only 1·3 lacs cusecs or about 7% is for irrigation. Attempts are now being made to use as water as possible, by the introduction of the multipurpose ı schemes, thus reducing wastage.

(3) **People**: Regarding population, there are enoug to do the work, it is needless to say that India is an agric country from the very beginning and there are 350 million in India now, of whom 70 to 80 per cent are agriculturists, s to develop irrigation in India is not a difficult job. This i the present Government is doing.

Going further into details:

North India: Here, the three big rivers with their tril are coming from the Himalayas. The rivers are perenr canals are taken directly from them and the vast pla irrigated. The plains in Uttar Pradesh (U. P.), Bil Bengal are irrigated by the Ganges and in the Punjab a by the Indus.

South India: This consists of hilly portions and Storage reservoirs or tanks are constructed and water from them by canals to irrigate the lands.

Plains: In this area anicuts or weirs are built acr to raise the water level, and canals are taken from them tc lands, by gravitation.

And throughout the country, there are, in additio tube wells in North India and ordinary wells in Sou These wells tap the subsoil water (which would othe without any use), which is used for irrigation by lifting.

Thus there is good facility for irrigation (by different throughout the country.

7. **Benefits of Irrigation:** (1) The main function of, and benefit derived from, irrigation is *the protection of the area from famine.* This is achieved both during the construction of the irrigation work and after its completion. During the construction of the irrigation work, if the area is famine-stricken, a good lot of labour available in the locality is engaged on the works and paid for, so as to provide their livelihood. When the work is completed, the area is generally irrigated and the people get the food they require.

(2) *It improves the condition of the people* by promoting trade and developing their resources. The value of their land is also enhanced, as the people can grow commercial crops which are remunerative.

(3) *It increases the revenue of the State*; this is done directly by the increase in the land revenue or by levying water-tax or water-rate. Also due to the development of irrigation, and the general improved condition of the population, there is the indirect increase in the revenue from the different departments, such as the Post and Telegraphs, the Railway, the Income-tax and the Customs.

(4) *It provides hydro-electric power* which not only develops the industrial activities, but also gives stimulus to domestic or cottage industries and agriculture (in the shape of providing irrigation pumps, etc.). Incidentally, the hydro-electric power solves the problem of shortage of fuel.

(5) *It generally improves the quality of the crops*, as the water supply to the land is assured.

(6) *It helps navigation* through its canals which are made in some cases specially fit for the purpose, and thus increased facilities for trading are created. (This was neglected until now in India, but recently steps are being taken to improve the inland waterways in the country).

(7) *It raises the water level in the wells and nalas* in the neighbourhood and reduces the cost of lift.

(8) *It improves or creates facilities for water supply to towns* for which the required supply can be taken from the canal or the reservoir itself, as conditions permit. Incidentally, the cattle in the locality are also benefitted by the drinking water.

(9) ***Plantation*** : On the embankment of the canal, trees can be easily grown. These may be either shady trees, timber trees or trees yielding fruits. All these can be easily grown and maintained and the transportation of the produce can also be easily effected through the canal. This item is also a paying concern.

(10) It provides ***facilities for bathing*** for the people in the locality and also for general washing purposes.

(11) In some cases, ***it converts the brackish water of the wells into sweet water.***

(12) It improves the country as a whole ***by making it self-sufficient in food.***

8. Bad Effects of Irrigation : (1) Usually when irrigation is newly introduced in a place, the tendency of the people is to over-irrigate their lands and, as a result of this, water logging is produced and the lands become alkaline. Special steps will have to be taken to prevent this.

(2) If the control of water in a newly irrigated land is not properly effected, or the drainage is not properly arranged, water will stagnate here and there, which leads to *Malaria.* This aspect should also be particularly managed during the execution of the project work.

(3) Due to irrigation, ***the climate becomes colder and damper,*** and this may lead to the ill-health of the people in the locality. This should be guarded against.

9. History of the Development of Irrigation in India : Agriculture by irrigation antedates recorded history and is probably one of the oldest occupations of civilised man.

(*a*) **Monuments of works done from the beginning** : The value of irrigation as a means of making the land more fertile was recognised even in ancient times, and there remains evidence of its use in India, Assyria, Babylon, Egypt, Persia, Italy and Spain.

India is an agricultural country from the very beginning and continues to be so even today. About 70 to 80 per cent of the population is still depending upon agriculture only. We see references to irrigation works in the oldest records of the Vedas, Smritis and Puranas, and coming to the period of recorded history itself. some stray examples here and there are noted below :

In Northern India, Chandragupta Maurya created facilities for irrigation in his empire, and in South India we see the Chola Emperors improving irrigation in the Cauvery delta. Then during the Muslim rule, Emperor Ferozshah Toghlak, in the 14th century, constructed canals from the rivers Sutlej and Jumna for the benefit of the people and we find that Akbar and Shahjahan improved these canals and also opened new ones. In Mysore during the 18th and 19th centuries, a good number of tanks were built and important weirs, Chikkadeva-rajasagara, Madhavamanthri anicuts were also constructed. In the 19th century, the British remodelled the old canals in Northern India and in the Cauvery delta, and constructed new canals such as the Upper Bari Doab canal, the Ganga canal and the canals in the Godavari delta

It will not be out of place to give here an extract from the speech of the Hon'ble Sri. C. Rajagopalachari in the International Engineers' Conference, New Delhi, on 11th January 1951.

"The International Engineers gathered here will find good enough proofs of genius and professional zeal in the irrigation works of India. The Cauvery Delta System in the South was originally planned and executed under Tamil kings who ruled 1700 years ago. The great dam of Bhopal was built nine hundred years ago by a Hindu King. The Jumna canal was built during the time of the Moghul Emperors. All these have been maintained and improved in recent times. In the British period the names of Sir Arthur Cotton, Sir Thomas Cautly, Lieut. Dyas, Captain Orr, and other well-known names are associated with the Great Irrigation works in India which they took up in the old tradition, inspired by what they saw and admired, and which in many cases were *improvements of more ancient works.* There has been a great and *sustained tradition of Irrigation Engineering in India*".

(*b*) **Recent Developments**: To improve irrgation, an Irrigation Commission was appointed in 1901 to examine the possibilities of improving the irrigation in the country. This gave good impetus and we have, as a result, many irrigation works in the shape of dams, barrages, diversion-weirs and canals.

Quite recently, the Union Government has established a body of experts called the Central Board of Irrigation and Power, and the Central Water and Power Commission, and these are authorised to look after the development of irrigation and advise and help the

State Governments. As a result of this, lots of projects (multi-purpose projects) have been prepared and at present they are in different stages of progress.

Again a good number of research stations have been established in many States; and finally the Planning Commission has included all important irrigation works in their Five Year Plans, and *topmost priority* has been given to irrigation works. India has thus doubled its fame, by maintaining its high tradition in Irrigation Engineering.

***India leads in irrigation as usual even now*:** It cultivates 266 million acres of which 52 million acres are irrigated. It is the oldest and biggest irrigated country in the world, and one can find all types of irrigation works in it.

In June 1950, India sponsored the establishment of an International Committee on Irrigation and Drainage, in a meeting held at Simla. Again in June 1951, India became the host country for the first International Congress on Irrigation and Drainage. Thus we see that ***India takes the first rank in irrigation in the World.***

Questions.

1. Describe briefly the necessity for, and benefits derived from, irrigation works in India. What are the ill-effects of excessive irrigation? **(A. M. I. E. Nov. 1952)**

2. Discuss the necessity and importance of irrigation in India. What are the advantages and possible disadvantages of irrigation?
(Gujerat Univ. April: 54 S, E. Civil)

3. "The aim of the life of every man, in whatever walk of life he may be, is to love and serve his living gods (i. e. human beings) as best as he can". What opportunities has an irrigation engineer got to fulfil this great law of life? As an irrigation engineer, what preventive and curative proposals would you give to mitigate the hardships of your fellow brothers living in the famine affected and flood affected parts of your country?
(Gujerat Univ. Nov. 53 B. E. Civil)

CHAPTER II

SOURCES OF WATER

1. Rain is the chief source of water required for man, but it cannot be got whenever required; and hence it is made use of in other forms, storing it when available.

When rain falls on the surface of the earth, a part of it is absorbed by the soil or vegetation, another part sinks deep into the ground, a portion is returned direct to the atmosphere by evaporation (and is lost once for all) and the remaining part runs off the surface, to the drainage lines forming streams or rivers and lakes.

2. **Classification of Natural Supply of Water:** Thus the required supply of water for use is got in one of the following forms:

(1) Rain

(2) Ground water supply:—(*a*) Wells, (*b*) Borings.

(3) Surface water supply:—(*a*) Streams or rivers, (*b*) Snow, (*c*) Lakes and (*d*) Springs.

(4) Sea and atmosphere.

1. Rain: Direct use of rain for irrigation can be made efficiently, when the rain is sufficient for the crop and is distributed throughout the crop season, enough to mature it. If the rainfall is 40 in. and above, and is also properly distributed during the whole of the period, the usual rainfall is itself enough. This is called ***natural irrigation***.

2. Ground Water Supply: The amount of water entering the ground depends upon the following factors:

(i) The amount of rainfall in the locality.

(ii) The capacity of the surface to drain off the rainfall to the nearest valley or stream.

(iii) The porosity of the surface.

(iv) The permeability and topographical features of the underground strata.

(*a*) *Wells*: Rain water that soaks into the ground is deposited and held there, when there is an impervious stratum below. This absorbed or stored water, is sometimes recovered again for use by excavation of 'wells'. When this water-bearing stratum is available within a short distance from the ground surface, and if the porous stratum above it is composed of coarse material, a well is easily sunk at such a place. (Vide Fig. 1).

WELL

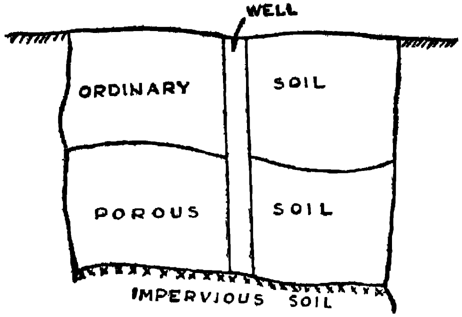


Fig. 1

(*b*) *Borings*: (i) When the water-bearing stratum is overlaid by impervious material of great thickness, boring is resorted to, and by piercing the overlying impervious strata by means of drilling machinery, tube wells are got.

(ii) If when piercing the impervious strata, there is an outlet for the confined water, and if the accumulated pressure is great, this water rises to the surface and forms an *artesian well*. When the rock is syncline, it is favourable for an artesian well and the water is easily tapped at the centre. (Vide Fig. 2).

ARTESIAN WELL

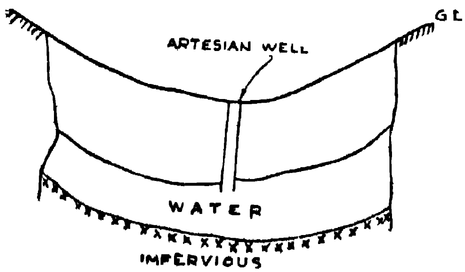


Fig. 2

(iii) Borings instead of being made vertical, are sometimes made either horizontal or slightly sloping. This kind of boring is called technically "tunnelling". The work done here is just like that for an ordinary tunnel, which is driven in sloping side of hill

country, to tap the subterranean water-supply. Shafts are first sunk at about 50 to 80 ft. apart, and the bottom of the shaft is connected to the tunnel and the excavated stuff is lifted up through the shafts. These are practically horizontal wells, differing from the ordinary wells, only in that the water has not to be pumped up to the surface but finds its way by gravity flow to the lands on which it is to be utilised. These tunnels are liable to be choked up and should be cleaned often. It is advisible, therefore, to provide a perforated lining to prevent choking.

Near the Kojak Pass in Baluchistan, there is a big tunnel of this kind. Here the water is used, not only for irrigation, but also for water supply. This is called by the name "**Karez**".

(For further information about Karezes, see chapter VII on Bandhara and other type of irrigation).

(iv) *Infiltration Gallery*: Ground water supply source is also in the shape of infiltration gallery. These galleries are generally laid in river beds where the depth of sand is small and not deep enough for wells. Loose pipes are put in at this depth (which is generally taken as 8 to 10 feet below the lowest water level) to intercept the underground water.

INFILTRATION GALLERY

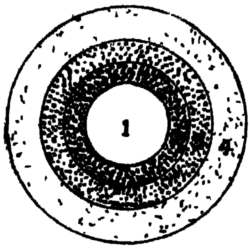


Fig. 3

The pipes are surrounded by inverted filters (Vide Fig. 3) to prevent sand and silt getting in and choking up the pipes.

They are placed (1) mostly across the line of flow of the underground-current of water. (2) They are also placed, sometimes, along the bank of the river and parallel to it to intercept and tap the underground flow. The advantages in this method are:

(a) The velocity is uniform.

(b) The pipe line can be extended when required to any length to augment the supply.

The disadvantages of placing the gallery at right angles to the river flow are:

(a) It is not possible to extend the gallery and thus get more supply.

(b) There is the risk of the gallery being washed away during floods in the river.

3. Surface Water Supply: (a) Streams and rivers: Natural water supply got by rain cannot always be used in the immediate vicinity, but it is stored in the shape of reservoirs, etc. from where it has to be transported to the places required.

Artificial transport is quite necessary, if the irrigation water is to be taken from a river or a reservoir. This transport or conveyance of water is effected by means of canals.

(b) Snow: Snow is a great storage of water, especially in North India. All the big rivers, Ganga, Brahmaputra and Sindhu get their water during monsoon from rainfall, but during summer, the snow on the mountains melts, and huge quantities of water are released into the rivers and thus there is good flow throughout the year.

(c) Lakes or Reservoirs: These may be either naturally formed in the shape of lakes, or artificial ones in the shape of reservoirs or tanks. If the lands to be irrigated are nearby, the water is led off directly from them to the lands, but if the lands are farther away, a canal or channel is excavated to convey the water.

(Reservoirs and canals are dealt with in full in Chapters VIII and XV pertaining to them).

(*b*) Springs: Springs are formed as a result of water under pressure between two impervious layers of rock or clay sandwitching between them pervious layers of soil.

Rain water percolating through a pervious stratum descends, until its downward motion is arrested by another underlying impervious stratum and then it follows the inclination or dip of the other stratum to the lowest point of its outcrop where it emerges as a ***spring***. (Vide Fig. 4),

SPRING

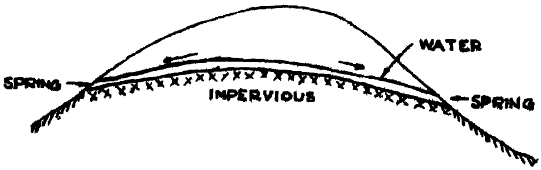


Fig. 4

4. The Sea and the Atmosphere : The sea is the greatest main storage, but the water here is saltish and unfit for irrigation; also the level of the sea does not permit cheap irrigation.

The atmosphere is also a great store of water, but water from the atmosphere cannot be easily and cheaply got. Hence these two sources, the sea and the atmosphere, are not taken into account here.

Questions.

1. What are the sources of underground water supply for irrigation ?

(Gujerat University, April 1953)

2. Write explanatory notes on : Karez.

(Gujerat University, April 1953-54)
(Poona University, 1953)
(Bombay University, 1949)

CHAPTER III

SOILS

1. Introduction : Soil is the fine material lying on the surface of the earth and it consists mostly of minerals from disintegrated rock and organic matter. Soil is a mixture of irregularly shaped particles of colloids, clay, silt and sand. The pore spaces in the soil vary from 34 to 44 per cent by volume. They act as a storage reservoir for the moisture to be used by the plant. The proportion of colloids materially affects the soil moisture property.

The soil consists of spheres, all of the same radius, packed together in a systematic manner. It is a heterogeneous mixture of the particles of clay, silt and sand.

Soil Horizon : This is usually divided into 3 horizons or parts :

Horizon A : Here the materials are decomposed and transferred into colloids and any compounds, if soluble, are dissolved. This zone is called the zone of eluviation or leaching and decomposition. The thickness of this horizon is about $1\frac{1}{2}$ ft.

Horizon B : The substances dissolved in the above zone are precipitated here. This is the zone of accretion. It is called the horizon of illuviation or deposition. The thickness of this horizon is about 3 ft.

Horizon C : This is the zone made up of decomposed rock or alluvium which is freshly deposited. The thickness of the zone is about 6 ft.

For sustaining plant life, five things are essential, namely soil, water, air, sunshine, and climate so the study of the soil in Irrigation is very important.

The suitability of soil for cultivation depends upon the *Chemical character* and on its *Mechanical properties* also. A plant requires good depth of soil to enable its root system to develop, and this depends upon the kind of plant. For instance, timber trees require a depth of 15 to 30 ft., fruit trees 10 to 15 ft., grain crops 4 to 5 ft. and ordinary garden crops $1\frac{1}{2}$ to 3 ft. (If the root of the tree cannot go deep, it spreads sideways).

2. Classification : Soils may be classified according to

(i) age,

(ii) the mineral contents (calcium, aluminium, iron, etc. which the soils contain),

(iii) the process of geological formation,

(iv) the size of the soil particles. (The Bureau of Soils in the U. S. A. has seven categories of soils from research work).

Mechanical analysis of soil is the quantitative determination of the particle sizes in the soil, as shown below:

Gravel : Soil particles larger than ¼" in thickness.

Sand: Soil particles between ¼" and 0·05 m. m.

Silt : Soil particles between 0·05 and 0·005 m. m.

Clay: Soil particles below 0·005 m. m.

(Soil particles below 0·02 m. m. are classed as clay, by the Irrigation Research Institute, Lahore).

Colloids : Soil particles less than 0 001 m. m.

Colloid is a gelatinous or gluey substance found in clays, especially of a sticky nature. It is an uncrystalline semi-solid substance and is capable of very slow diffusion or penetration. The diameter of the grain or particle is less than 0·001 millimetre and as such, it may be said that if the particles in a soil are very fine, the soil is colloid. The colloid absorbs moisture and so, if a soil contains more of colloids, it has greater moisture.

Soils may also be classified as Alluvial and Non-alluvial.

3. Alluvial soil occurs in Assam, Bengal, Bihar, Orissa, Utter Pradesh and the Punjab ; that is in the Gangetic plain, the Indus plain, deltas of rivers, and near the coast. The alluvial soil is divided into (a) recent alluvium and (b) old alluvium. For example the first occurs on the north bank of the Brahmaputra, and the second on the south bank of the Brahmaputra in Assam.

Alluvium varies from rich loam to brown sand. The inclination of the surface of alluvium is very gentle.

Advantages of alluvial soil : (1) The rainfall is easily absorbed in the soil which thus becomes a reservoir for underground water.

(2) As the country is also flat where alluvial soil is abundant, large projects are feasible.

(Sometimes salinity occurs in the alluvial soil, and in such a case, it cannot be of much use for the crops).

Non–Alluvial soils

4. **Red soil** is crystalline and of sandstone formation. It occurs in broken and undulated country; no big irrigation projects are feasible in this area. Where this soil is occurs, only tanks or reservoirs can be constructed.

Red soil is light-textured, porous and friable; hence it is good for cultivation. But the soil varies in fertility from place to place. It is generally acidic or neutral (and not alkaline). It contains some lime and phosphate. There is no free calcium (kankar) in it.

Laterite occurs usually on the summits of hills. It is predominant in Orissa, Madhya Pradesh, Travancore, the Deccan and Central India. The laterite soil lacks the common nutrient, such as potash. It occurs in regions of copious rainfall, as in Malnad in Mysore and in South West parts of Bombay, and so it is good for irrigation.

5. **Black soil** : The depth of black soil varies from 1 to 2 ft. to even 20 ft. The texture of the soil varies from clay to loam. The characteristic of this soil is, that it develops cracks due to heat of the sun. It contains uncombined calcium carbonate. The soil is rich and productive. As the soil is heavy, it is good for growing sugarcane.

6. **Clay pan** : The soil here is not cemented. The clay pan layer is largely formed by the accumulation of impervious stiff and compact clay.

7. **Hard pan** : Dissolved materials like silica, calcium carbonate, when precipitated at a particular depth, form the hard pan. The soil does not allow the root of plants or water to penetrate into it easily.

Note : General formation of pans : Year after year, layers of calcium, magnesium, iron, etc., are formed in soils rich in humus when there is no free drainage. This is called **pan**.

8. **Peet** : Organic materials varying from 20 to 90 per cent of their total weight abound in some soils. These organic materials

are formed by the decomposition of plants in marshes, or bogs, and the resulting soil is called **peat.**

9. **Humus** is an essential element of soil fertility. It affects the moisture-holding capacity of the soil to some extent. The brown or black powder in which the soil is formed by the reaction of air on animal or vegetable matter is called **humus.**

10. **Sandy soil** : This is available all along the sea coast and in a large area of Rajasthan.

11. **General** : **Alluvial soil** occurs in plain and even country. Where this is met with, proper foundation for the masonry structure should be provided with care. The rivers in this area are shallow and sometimes change their courses also. The bed of a river in alluvial soil consists of a very thin layer of soil, unlike the bed of a river in a hilly country. For example, the Indo-Gangetic plain, the deltas of the rivers Ganga, Mahanadi, Godavari and Cauvery.

Non-alluvial soil: occurs by the disintegration of rock for a very long time. Where this soil is met with, the river runs in a valley and the ground is mostly rocky and hard foundations for any masonry structure can be got easily.

12. **Soils in the Bombay State :** The soils generally met with in the Bombay state are coastal alluvium, medium black soil, deep black soil, or trap, clay loam (black) sandy loam, coarse sandy soil, coarse shallow soil (or granite gneiss), mixed red and black soil and laterite.

13. **Soil Maps :** Soils are marked on irrigation maps in different colours to distinguish them one from another. The usual practice is as noted below :

(1) Barren or waste lands :—Yellow ochre.
(2) Light soil (which can be improved) :—Burnt sienna.
(3) Good soil (fit for irrigation) :—Light sepia.
(4) Best (fertile) soil :—Dark sepia.

14. **Physical Properties of Soils:** (i) *Porosity*: This is the ratio expressed as a percentage of the inter-granular space in a given soil sample to the total volume occupied by it. This varies with the texture of the soil. It is larger in soils having a large percentage of silt and clay. For cultivated soils, it is 15 to

35 per cent, for sandy soils it is 20 per cent, for clayey soils it is 50 per cent.

(ii) *Permeability*: It is the capacity of water to percolate through soils. It is a measure of the quantity of water passing through unit cross section in a unit time with 1 to 1 hydraulic gradient. It varies as the square of the diameter of the grain of the soil, the ratio of fine material, and with the arrangement of the grains.

(iii) *Moisture Content*: Moisture content or water content of the soil is the ratio expressed as a percentage of the weight of moisture contained in the soil, to the weight of dry soil. This is got by finding the loss in weight, when a weighted quantity of wet soil is dried to a constant weight, at 105 to 110 degrees centigrade. This loss in weight represents the weight of the water and it is expressed as a percentage of the weight of the dry soil mass.

(iv) *Capillary rise*: In coarse grained soil, the time required to reach the limit of the capillary rise is much less than in fine texture. The capillary rise or lift in grits is from 5 inches to 10 inches, in sand it is 1 to 4 ft.; in finer soil it is even 5 to 10 ft. To complete the full capillary rise, the time taken varies from 3 to 12 months.

(v) *Specific Gravity*: The specific gravity of some soils is given below :—

Soil		Specific Gravity
Fine sandy loam	...	1·15
Silt loam	...	1·31
Clay loam	...	1·36
Clay	...	1·59

N. B. Weight of soils per cubic foot:

Soil		Weight per cubic foot
Clay	...	75 to 80 pounds
Loam	...	85 to 90 pounds
Sandy loam	...	90 to 95 pounds
Sand	...	95 to 105 pounds

15. Chemical Characteristics of Soils: Here only two things, namely, the salt contents and the pH values are taken into consideration as they determine the crop to be grown on the soil.

(a) Salts: The salts (alkaline salts) met with in any ordinary soil include chloride, sulphate, carbonate and nitrate of calcium. The Calcium carbonate and sulphate are not sufficiently soluble in water and hence they are not harmful to the crop. The food nutrients used to the soil are Sodium and Potassium nitrates. These, though beneficial as manure, are harmful if they are used in excess quantity. The three harmful salts in the soil are sodium carbonate, sodium chloride and sodium sulphate. The other salts are mostly neutral and are not harmful.

Injurious salts: These, as said above, are harmful to the soil, and of these, sodium carbonate is the worst. It is a white salt, and by decomposing the organic matter in the soil, it produces a black patch on the surface and hence it is called "black alkali". The sodium chloride and sulphate produce white alkali.

The yield from the soil is not generally affected up to, say, 0·18 per cent of the total salts. If the percentage exceeds 0·25, the soil becomes useless.

The harmful salts in a soil should not be more than

0·10 per cent in the case of sodium carbonate,
0·25 ,, ,, ,, sodium chloride,
0·50 ,, ,, ,, sodium sulphate.

(b) pH Value: pH value is the degree of alkalinity or acidity in a soil. It is given a value ranging from 0 to 14. Of this 7 indicates neutral soil. Below 7 the soil is taken as acidic, and above 7, alkaline.

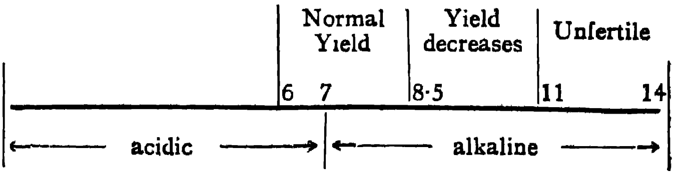


Fig. 5

With pH value

as 6 or 7 to 8·5, the soil gives normal yield or crop,
as 8·5 to 9, the yield decreases,
and above 11, the soil becomes unfertile.

16. Soil Water: This is the water held in the upper portion of the zone of aeration and it is limited only to the surface layer. The depth of soil water varies very considerably; it may be only a few inches or sometimes may go to a depth of even 50 ft. It is practically a reservoir for the plants to derive their waters from, for sustenance.

17. Gravitation water or free water is the water in excess of the hygroscopic water and capillary water. It is free to move downwards when there is good drainage available or when the soil is porous. This occurs below the water table and extends downwards up to the first impervious stratum.

18. Soil Moisture: Soil moisture can be divided into three heads:

(1) Available moisture.
(2) Unavailable moisture.
(3) Residual moisture.

(1) *Available moisture* is the moisture available between field capacity and Wilting coefficient. This is subdivided into (a) readily available, and (b) less readily available moisture.

(2) The *unavailable moisture* is divided into (a) moisture occasionally available with difficulty (this occurs between Wilting Coefficient and the hygroscopic coefficient) and (b) completely unavailable moisture (this is below the hygroscopic coefficient).

Soil Moisture Content: The equilibrium quantity of moisture in the soil at different conditions is known as soil moisture content. The percentage of moisture in the soil representing the content is given on the dry weight basis. (Vide Fig. 6).

SOIL MOISTURE CONTENT

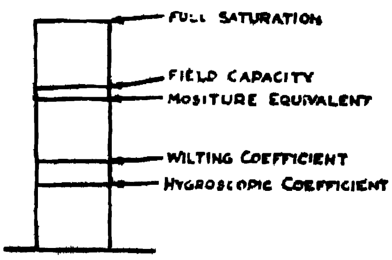


Fig. 6

19. Hygroscopic Coefficient: Hygroscopic moisture or coefficient is the percentage of moisture which a soil is able to retain when air-dried, or the percentage of moisture an ovendry soil is able to absorb from saturated air, at a constant temperature. This moisture, though not much useful to plants, should be present in the soil before it can contain useful portion of the soil moisture.

20. Wilting Coefficient: The permanent Wilting Coefficient is the percentage of moisture in the soil at which plants will wilt permanently unless water is applied permanently.

When the rate of transpiration exceeds the rate at which they can absorb moisture from the soil and transmit it to the leaves, the plants may droop. The Wilting Coefficient is generally independent of the crop on the soil, but varies with the soil texture.

Wilting coefficients usually assumed are:—

Soil		Wilting Coefficient
Sandy loam	...	9·66
Fine sand	...	2 59
Clay loam	...	16·30

If the moisture is less than that required by the plant, it first droops and then wilts. Even when there is sufficient moisture in the soil, during the hot portion of a day a plant may droop. When the plants do not recover even during nights (when there is less transpiration) and wilt, water should be added to reduce the turgidity of the plant. At this stage, the plant roots extract more water than the soil grains can hold.

A definite relation exists between the wilting coefficient and the moisture equivalent:

$$\text{Wilting Coefficient} = \frac{\text{Moisture Equivalent}}{1{\cdot}73 \text{ to } 3{\cdot}82}$$

To make effective use of the moisture content of a soil as a guide to the proper time when irrigation should be done, one should know how to apply the wilting coefficient of the soil.

Plant roots exert a pulling on the soil moisture. If the moisture is less, the root gets less supply, and at a certain stage when there is no supply, the plant wilts. The moisture content at this stage, that is, when the plant cannot get its supply, is called the *Wilting point*. It is the moisture content at which the permanent wilting of the plant occurs.

Experiments conducted at Colarado have shown the following values for the wilting coefficient:

Soil		Wilting Coefficient
Coarse sand	...	less than 1
Fine sand	...	2·5 to 3·5
Sandy loam	...	5 to 6
Loam	...	10 to 15
Clay loam	...	14 to 15·5

21. Moisture Equivalent: Moisture equivalent is the percentage of water retained by a test sample of the soil.

Moisture equivalent is generally taken as

= field capacity, or

= 1·8 to 2 times the wilting coefficient, or

= 2·7 times the hygroscopic coefficient.

Moisture equivalent for certain soils is noted below:

Soil		Moisture Equivalent
Sand	...	3·6 per cent
Fine sand	...	4·3 per cent
Sandy loam	...	10·3 per cent
Fine sandy loam		12·3 per cent
Silt loam	...	19·8 per cent
Clay loam	...	23·1 per cent

Centrifugal Moisture Equivalent

When a soil, whose moisture equivalent has reached saturation point, is subjected for an hour to a centrifugal force equivalent to 1000 times the gravitation, it is called Centrifugal moisture equivalent.

22. Minimum Moisture: As the supply of moisture in a soil approaches the minimum, the plants begin to secure this moisture with great difficulty and at this stage the growth of the plant is affected. The point, at which this moisture is the minimum, and the water or moisture has to be replenished, is called the minimum or *optimum moisture.*

Roots of plants require both air and water. In a saturated soil there is no air and in dry soil there is no water. Hence the soil for plant life should be midway between the two.

23. Soil Texture: The size and gradation of the particles in a soil indicate the texture of the soil. The texture depends upon the clay content of the soil, and according to the clay contents, soils are divided as follows:

(1) *Light soil*: Clay contents 2 to 8 per cent—fit for growing grain and fodder crops.

(2) *Normal soil*: Clay contents 10 to 20 per cent—fit for cotton, wheat and maize.

(3) *Heavy soil*: Clay contents 40 per cent and above fit for sugarcane and rice.

If the contents are less than 2 per cent, the soil is useless. Texture of the soil determines the porosity of the soil which is larger in soils having large percentage of silt and clay, and the quality of the crop grown on any soil depends upon its texture.

Red soil is light-texture (porous and friable); the texture of black cotton soil varies from clay to loam.

24. Soil Structure: The manner in which the soil particles are arranged (in groups) is called the structure of the soil.

25. Tilth: Tilth is proper cultivation of the land. If the arrangement of the soil particles in relation to one another is satisfactory, the land is said to have good tilth. This is maintained by proper ploughing and thus loosening the soil. Tilth prevents loss of moisture in the soil by evaporation and it also increases the yield of the crop by the cultivation and the consequent proper æration of the subsoil.

26. Preparation of the land for Irrigation: All lands required to be irrigated are not usually regular in form and smooth. As such, they should be made to fit in for the intended irrigation, so that the required water may be uniformly distributed over the entire land.

For this purpose, the following items will have to be attended to:

(1) Clearing the jungle and grubbing the roots.

(2) Levelling and grading the land.

(3) Dividing the land into different plots.

(4) Excavating the required distributory channels or water courses.

(1) *Clearance of Jungle etc.*: This item includes not only removing the small jungle, brush, and sods, but also removing trees. The small jungle and sods are generally removed by ordinary ploughing, pulling and raking, and sometimes by burning in addition.

For removing trees, they are first cut to the ground level in the ordinary way, but the stumps will have to be removed separately. Small stumps varying from 6 to 8 inches in diameter are economically removed by hand grubbing, and the big stumps from 2 to 8 ft. in diameter are removed by burning.

(2) *Levelling and grading the land*: The amount of grading and levelling depends upon the character of the soil, the topography, the method of irrigation and the area to be prepared.

The object of the work is to see that the water is applied smoothly throughout the land. All the mounds in the land should be removed, all depressions filled up and the whole land levelled. It should be graded also, that is, given a slope, so that the water may flow properly to the entire area of the land.

Tractors and scrapers are used to level the area, to begin with, and for the final finishing of the surface, various forms of floats are used.

(3) *Division of the land into plots*: This work is done so that the irrigation water is confined in small areas and distributed uniformly throughout the land. The division of the land also depends upon the method of application of water.

(4) *Excavation of Distributary channel or water courses*: The distributary channel or water-course is to be properly aligned with a gentle fall and distributed throughout the entire area of the land. For getting on with this work, a careful plan should be first prepared of the area to be irrigated, and the levels (original and proposed) marked thereon and then the proposals worked out. It may be said, in this connection, that the cost of preparing a map or plan is worth its expenditure.

Even when the land is made fit for irrigation, there are various methods of applying water to this land: and the preparation of the land depends upon the particular method of application of water. These methods are dealt with in a separate chapter.

Soil Erosion and Conservation

27. Soil Erosion and Conservation: Soil particles are removed from their original resting places by some external agencies, like wind or water. This process is called *soil erosion.* By this process, the soil loses much of its manurial soil particles, thus becoming useless for productive agriculture. The process by which the agricultural land is prevented from soil erosion is called *Soil Conservation.*

28. Causes for Soil Erosion : The natural causes for soil erosion are

(1) Near the foot of the hills, the surface slope is very steep.

(2) In these places, rainfall is also heavy, owing to which deep scours are formed.

The above causes for soil erosion are mainly due to wind or water, and are natural.

Soil erosion also takes place from other causes as detailed below :

(i) The way in which the land is ploughed and the kind of farming :—If the land is not cultivated properly, it weakens the resistance of the soil, then soil erosion is apt to occur. The faulty methods of cultivation are

(a) Excessive use of fertilizing manures,
(b) Lack of proper ploughing,
(c) Not adopting the method of rotation of crops, and
(d) Improper rotation of crops.

(ii) The removal of the cover of grass and forests from the ground, to bring it under cultivation, makes the ground bare and uncovered, thereby giving rise to soil erosion.

(iii) Grazing is not controlled in many places.

29. Classification : There are four types of soil erosion which are described below :

(1) *Water Erosion :* Water erosion is more common and is due to the flow of water on the surface of the land, with a high velocity. This is either due to the high velocity of water itself or the steep slope of the land.

(2) *Wind Erosion*: As already said above, this is due to strong winds by which the soil particles are carried away from their original resting places.

(3) *Sheet Erosion*: Here, by the action of heavy rain, the crust is uniformly (like a sheet) removed throughout the area. This procedure or process is insidious and cannot be noticed at all for a long time and the fertility of the soil gradually decreases year by year.

(4) *Gully Erosion*: In this process, water flows in defined channels and as the eroding effect increases, it results in the land being made in the form of trench-like gullies, which become deeper and wider as time passes. Contrary to sheet erosion, gully erosion, can be easily detected. If this happens in fields, reclamation is costly.

30. Disadvantages of Soil Erosion or Damages done by Soil Erosion: The classes of soil erosion, described above, give us some idea about the damages done by it to the soil. The damages done by erosion are given below:

(1) Soil erosion results in not only the loss of soil but also in loss of fertility of the land, and thus the land becomes unproductive.

(2) Owing to the unproductiveness of the land, animals are under-fed and are not nourished well. The people will also have to face food problem.

(3) The land loses all its beautiful attractive landscape and its value is also reduced.

(4) With the deforestation, rain water flows down the bare land with high velocity, destroying animal life.

(5) There is loss of water in the shape of run-off, and this results in droughts and flood. Hence floods have to be controlled to prevent damages to crops from them.

(6) There is silting of canals and streams, which affects Inland Navigation.

(7) Deforestation generally reduces the rainfall in that locality.

31. Soil Conservation or **Control of Soil Erosion**: The following methods are adopted for controlling soil erosion:

(1) *Terracing*: The excess of water on a field is made to flow at a low velocity by building flat fields with a small longitudinal grade. Each terrace reduces the velocity of water, itself acting as a shallow diversion ditch.

The disadvantages of terracing are the following :

(i) It is costly to be built.

(ii) It is difficult to maintain.

(iii) It leads to changes in farm practice (which is not appreciated by the cultivators), and

(iv) There is damage to the land, due to the diversion and concentration of water at unnatural exits.

(2) *Contour Farming*: In this system, the rows of bunds put up act as miniature terraces; they hold the rain water, increase the absorption and thus reduce the run-off. (If the rows are formed with slopes, intermediate furrows act as channels and gullies are the result).

(3) *Contour Bunds*: These are small earthen bunds varying in size from 1 to 3 ft. in height and constructed on the contour of the ground. These are commonly seen in the dry districts of Chithradurga, Bellary, Bijapur and Dharwar (Mysore state), where the rainfall is sudden, heavy and fluctuating. On the topo sheets of these districts, the disposition of these bunds is clearly marked. The advantages of these bunds are two-fold :—

(i) They prevent scour of the soil, and

(ii) Water is stored and the land is made richer (to grow the *Jola* crop).

The construction of these bunds is an item of the "Grow More Food" Campaigan, launched by the Government of India and a good amount is spent for the conservation of soil by this method.

(4) *Plants Growth*: Some kind of plants is allowed to grow on the field at all seasons. The plants hold the water and reduce the flow and so give skin protection against the sheet flow. As the same plant cannot be on the field during all seasons, crops are rotated giving minimum period between them.

Afforestation should be introduced on the slopes of hilly areas. This is called *Vanamahotsava*, introduced everywhere compulsorily (by the Government) for the benefit of the people.

Note: Deforestation must be stopped by introducing special laws.

(5) *Temporary Dams:* These are put up against streams with (i) Brush wood (ii) Woven wire and (iii) Logs.

Brush dams are less permanent. They are tedious to be built. Woven dams are check dams which are adopted to V shaped gullies with narrow bottom and small water sheds. Log dams can be constructed where timber is available in plenty.

(6) *Grazing should be controlled:* The worst havoc is done by goats. Even if any other animal is allowed, goats should be strictly prohibited. No grazing should be allowed wherever possible and if grass is necessary, it should be cut and taken for use.

(7) *Detention Basins:* For the prevention of erosion, either detention basins or rock fill dams are constructed. Even when water is intercepted in these dams, it is allowed to pass out gently through percolation. These should be emptied before each next rain. Generally small catchments only are tackled by this method. Branch torrents are dammed by rock fill dams where no weirs are necessary.

Recently research work by the special bureau of soil reclamation in Colorado, U. S. A., with Dr. Karl Tezachi, is being done in soil Mechanics to study the extent of soil erosion, the causes and the remedial measures for the same.

Questions

1. Write short notes on Saturation Gradient
(Mysore University March 1945 and 1946).

2. Write short notes on Wilting Coefficient.
(Bombay University April 1950, B. E. Civil).

3. Write short notes on Soil Moisture.
(Gujerat University April 1954, S. E. Civil).

4. Write a brief note on soil moisture. What is the importance of optimum water supply to soil, and the Wilting Coefficient?
(Gujerat University April 1953, S. E. Civil).

5. Write short notes on Wilting Coefficient. Distinguish between texture and structure of the soil.
(Gujerat University April 1955).

6. Explain the function and indicate the difference between Hygroscopic and gravitational Moisture.

(Poona University April 1955).

7. Differentiate between texture and structure of the soil. Draw a map of the Bombay state and show thereon the types of soils met with in the different regions. To what types do the soils in Gujerat region belong ?

(Gujerat University April 1954).

8. "Thou shalt not destroy indiscriminately the cover of soil". What are the bad consequences when man does not follow this warning from Nature? How can these consequences be avoided ? **(Gujerat University April 1954).**

9. Describe 'soil erosion' in detail and its various forms. How does it affect irrigation projects ? What methods are usually adopted to control erosion ? Give sketches where necessary.

(A. M. I. E. May 1953 and Gujerat University, April 1953).

10. Write what you know about soil erosion. What are the usual methods being adopted to check it ? How does soil erosion affect irrigation ? **(A. M. I. E. November 1952)**

11. Write short note on: Soil Erosion.

(Mysore University, March 1946).

12. What are the chief causes that lead to the silting up of tanks and what are the measures to be adopted to prevent it ?

How can you practically judge whether a tank is silted up or not, and how will you determine the exact reduction in capacity ?

Briefly explain the silt theories you know of.

(Mys. Univ. B. E. Civil Sept. 1956).

13. Describe the various forms of soil erosion and steps usually taken to control the same. How does this affect the irrigation projects ?

(Mys. Univ. B. E. Civil. Sept. 1956).

14. State briefly the physical and chemical properties of soils suitable from the point of view of irrigation and why do the soils become unsuitable ?

(Bom. Univ. B. E. Civil April 1955)

15. Write short notes on Optimum water supply.

(Bom. Univ. B. E. Civil April 1955)

16. Write explanatory notes bringing its usefulness in irrigation practice on : Wilting Coefficient.

(Bom. Univ. B. E. Civil April 1956)
(Guj. Univ. S. E. Civil. April 1956)

17. Write short notes on Soil Erosion and its control.

(Bom. Univ. B. E. Civil April 1956)

18. Why is a proper study and classification of soils necessary for establishing an efficient and successful Irrigation System ?

(Poona Univ. B. E. Civil April 1957)

19. Explain clearly and comment briefly on :

(i) In respect of Soil-Damage due to Irrigation, 'Prevention is better than Cure', is a good policy.

(ii) Light soils are not generally suitable for Rabi Irrigation. **(Poona Univ. B. E. Civil April 1957)**

20. (a) How will you determine the suitability of a soil for irrigation purposes ?

(b) What are the main soil types met with in Gujarat and what are their characteristics ?

(c) What precautions are required before introducing perennial irrigation in the above soils ?

5. What is the difference between: Flow Irrigation and Lift irrigation? In which parts of Mysore is lift irrigation practised?

(Mysore University, March 1947).

6. (a) What do you understand by 'Irrigated Dry Crops'? (b) Differentiate between Direct and Reservoir irrigation and discuss when you would adopt each of these types.

(Mysore University B. E. Civil, Sept. 1954).

7. Describe briefly the systems suitable for irrigation a tract of country from (a) a river with a constant flow of water, and (b) a river with an intermittent flow and subject to heavy floods. Draw sketches of the kind of head works suitable for each. Give examples of each kind you have seen.

(Mysore University B. E. Civil, April 1955).

8. Distinguish between Perennial Irrigation and Seasonal Irrigation. Give briefly the advantages and disadvantages of each.

(Poona University, April 1957).

CHAPTER V

RAINFALL—RUNOFF & DISCHARGE RAINFALL

1. Hydrography: Rainfall is the source of all water used for irrigation and hence a knowledge of its amount, character, season and the effects produced by it, should all be studied. *Hydrography* is the science which deals with the above.

Rainfall or mean annual rainfall is the mean rainfall for a year observed over a period which is sufficiently long to produce a fairly constant mean value. It is expressed as so many inches of depth of water over the area.

2. Natural Distribution of Rainfall: Of the rain falling on land a portion evaporates, another portion is absorbed by the soil, percolates into and goes underground, and a third part runs into stream or valley, to be used when required, either for irrigation or for water supply.

3. Variations of Rainfall: In England, where the total annual rainfall does not vary greatly, the following laws have been formed.

(1) *Glaisher's Law*: The average rainfall of the three consecutive years yielding the least fall is taken as the average rainfall.

(2) *Hawshley's Law*: From the average rainfall of twenty years, one-sixth of the average of three years of rainfall is deducted.

These two laws should agree in any particular case.

In India, such general rules have not been derived, or if derived, they cannot hold good, owing to the extreme variations that exist on the one hand between a famine year, when there may be almost absence of rain, or in a year of excessive rainfall when the precipitation may be many times that during an average year.

4. Causes for Variation: The mean annual rainfall in a country varies from place to place. Even in the same locality, there is variation from month to month, or from even day to day. There may be rain in a locality and just a furlong or two from it

there may be no rain at all. It is very difficult to account for these variations; and only a general idea giving the causes for variation is given below :—

(1) *Latitude*: As is well known, the rainfall in a colder latitude is more than in a hot place, because the clouds tend to condense and precipitate at the place.

(2) *Altitude*: Rainfall is usually greater at higher altitudes, because they are able to intercept or attract the clouds. It may be said also that such altitudes are cold, and as such will cause a heavier rainfall.

(3) *Proximity to high mountains*: It is generally seen that places near the mountains get more rainfall. In this connection, it is well known that there is more rain on the windward side than on the lee-ward side.

(4) *Proximity to sea*: Places near the sea coast usually get more rain.

(5) *Character and direction of the prevailing winds*: If a place is situated in the path of monsoon-carrying rainclouds, that place naturally gets more rain.

(6) *Proximity to Forests*: Forests tend to induce rainfall, and it is generally seen (though not scientifically proved) that areas near the forests get more rain than those in the plains.

Though two places may be situated very close to each other, but when there is an intervening hill, if the levels of the two places differ greatly, the rainfall in the two places varies much.

5. Isohyet is a line on a rainfall map showing places having the same average annual rainfall.

From the study of the rainfall registers of good many places in South India, it is seen that

(1) Fluctuations are smaller in larger areas than in smaller areas.

(2) If the rainfall is more, the difference or fluctuation is small and vice versa.

6. Characteristics of Rainfall in India: Rainfall in India, unlike in England, is not sufficient and uniform throughout the year. The chief characteristics of the rainfall here are,

(1) Unequal distribution of the rainfall during the season.
(2) Unequal distribution over the different parts of the country.
(3) Partial failure or sometimes complete failure of the rain.

There is greater precipitation of rain in the major portion of India during the south west monsoon, but in South India, the rainy season is in the north-east monsoon. Regarding the variations in the rain, or "vagaries of nature or rain" as it is called, the rainfall in India may be described as below:

Regions of heavy rainfall are the western coast to the west of the Western Ghats, and the foot of the Himalayas. This is due to the direction of the monsoon wind, and also to the nearness of both the sea and the mountains.

The rainfall in the west Punjab is 10 in. and in the East Punjab it is 40 in. This is in accord with the theory that the rains increase with latitude. Again, the rainfall in Saurashtra is 30 in. and to the west of it in Bikaner, it is about 5 in. This is due to the direction of the wind from the ocean affecting the precipitation, and though the rainfall on the west coast is roughly 100 in., just to the east of it beyond the Ghats, (for instance in Hyderabad and Deccan), it is about 20 in.; this is due to the Western Ghats intervening. As a further example of this theory, it is seen that though Palghat and Coimbatore are situated only about 30 miles apart, the rainfall at Palghat is about 78 in. and at Coimbatore it is only 22 in., because of a hill intervening.

About the quantity of rainfall in India, Sir Alexander Binnie has calculated that "if the averge rainfall of India (including Burma and the Himalayas) is distributed over the whole country, we could form a sheet of water about 42 in. in depth throughout the country." Details of rain in some special places in India are given below: Bikaner desert 2 to 5 in. and South of Bombay (west coast) 140 in. In Kassia and Jayantha hill (Assam) at the head of the Bay of Bengal, the rainfall is more than 150 in. At Chirapunji is is 426 in. (It is seen here, that there was a downpour of 30 in. in each of the five successive days and the maximum in 12 months was 900 in. in 1861). A rainfall map of India is given in Fig. 9.

7. Mean Annual Rainfall: From experience it is agreed that the mean annual rainfall at a place should be taken from the average for at least 35 years.

MAP OF INDIA SHOWING THE ANNUAL RAINFALL

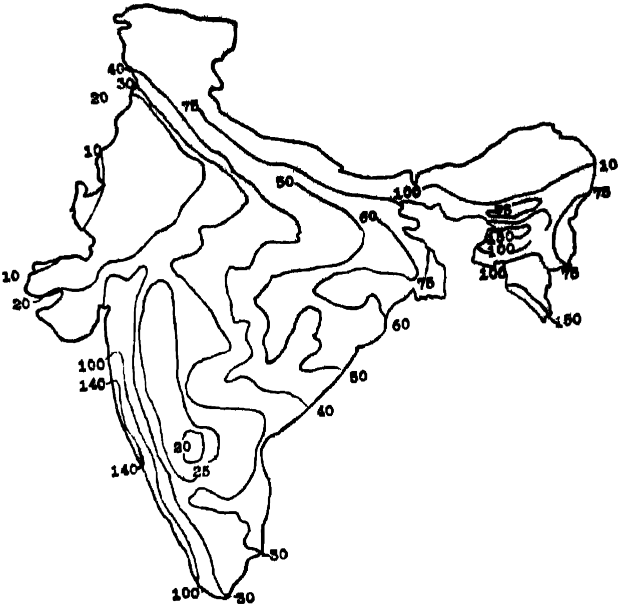


Fig. 9.

When such a record for 35 years cannot be got, a correction as noted below (given by Sir Alexander Binnie) is generally used.

Years	5	10	20	30
Correction	14·92	8 2	3 27	2·26

8. Average Rainfall of a Bad Year: From the registers maintained in the several rain gauge stations (in South India), it is seen that the rainfall of an average bad year can be safely taken as two-thirds to three-fourths of the mean annual rainfall.

9. Intensity of Rainfall: The maximum rainfall during a short period is called the *Intensity of rainfall.* This is very important, especially as it affects the flow from the catchment area.

10. Rain Gauges: The records of rainfall, or rainfall registers as they are called, are very important records, as they form the foundation of knowledge of the water resources of a country. In every district, Rain Gauge stations are established at important places. They are on the average at about one per 50 sq. miles. The Revenue Department mostly maintains all these stations. In some cases, the P. W. D. and in other cases the Forest, the Police or other departments also maintain these rain gauge stations, as the circumstances require.

According to the table given below, the number of rain gauge stations is generally fixed.

Area (sq. miles)	Number of Stations
0 to 50	1
50 to 100	2
100 to 200	3
200 to 350	4

The rain gauges should be locatod as follows:

(a) Good open spaces should be selected for the location of the rain gauge.

(b) The gauge should be so erected that the distance between it and the nearest object, should not be less than twice the height of the object.

(c) Within 30 yards of the gauge, there should be no tree.

(d) If a suitable site on a level ground is available, it should be preferred to the side or top of a hill, for the location of the rain gauge.

Note: Provision should be made for the protection of the rain gauge from winds, as it is seen that the wind forms an eddy over the mouth of the gauge, and carries away small drops which would otherwise have fallen into the gauge and that it registers consequently less than the true amount of rain.

Usually rain gauges are of two types (1) Non-Automatic rain gauges and (2) Automatic or self-recording rain gauges.

(1) Non-Automatic type: The type of rain gauge which is in general use throughout India is the Symon's Gauge which is of

the non-automatic type. It consists of three parts as shown in Fig. 10.

(i) A base which is built into a masonry, or concrete foundation.

(ii) A body in which the glass bottle collecting rain water is placed.

(iii) A funnel which collects the rainfall.

A 2 ft. cube of concrete or masonry forms the foundation of the gauge. It is sunk into the ground with its top 2 in. above the general level of the natural surface. The base of the gauge is

SYMON'S RAINGAUGE

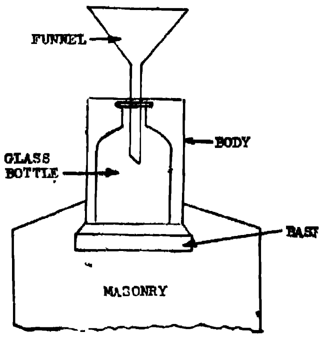


Fig. 10

firmly built into this foundation, so that the top of the completed gauge is one foot above the natural ground level.

Great care must be taken for the setting up of the gauge, so that the top of the funnel mouth is perfectly level, and to prevent its being blown off by wind, the base of the gauge should be set 4 in. inside the masonry. (See Fig. 10, 11). To protect it from damages if any, the base is generally surrounded by an open fence of such a size that the top of the fence is not higher above the mouth of gauge than half its distance from the gauge.

A cylindrical vessel 5 in. in diameter with its base enlarged to 8" in diameter constitutes the Symon's gauge. A cylindrical brass ring is provided on the top portion of the funnel the internal diameter of which is exactly 5" and the bottom of the funnel is led into

the neck of the bottle. A graduated flask is usually supplied, along with the instrument for the measurement of the rain water.

The rainfall is measured by pouring the rain water collected in the gauge to the measuring graduated jar provided for the purpose, as said above. The graduations on the jar are such that each division represents one-hundredth of an inch of rainfall. If there is more water than the measuring glass can hold, it should be carefully filled to the top graduation mark, and then this water poured away and the graduated glass refilled. The sum of all the measurements taken during the previous 24 hours constitutes the total

RAINGAUGE FIXED IN MASONRY

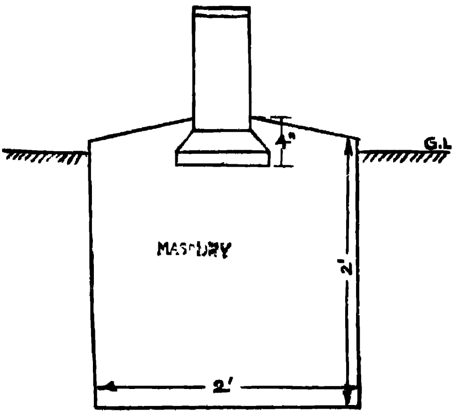


Fig. 11

rainfall for the day. Generally the capacity of the receiving bottle is not more than what 3 to 5 inches of rain will come to. Usuall, at 8 A. M. everyday, the rain water in the guage is measured. The day always counts from 8 A. M. of the previous day to 8 A. M. of the day in question.

N. B. Whether any rain falls or not, it is the duty of the rain-gauge recorder, to examine the gauge at that hour every day so that the gauge may be kept clean. When there is heavy rainfall (i. e. the intensity is great), the time during which the maximum rainfall occurred should be specially noted.

(3) *Using run-off Formulae:* Formulae formed by many experts to find the run-off from a catchment, are available for the purpose. Only a few of the formulae are given below:

(a) Inglis Formula: A formula for the run-off of a catchment is given by Inglis, derived from his experiments in the Bombay-Deccan catchment area. In his formula, he has taken into consideration two types of areas (i) The Ghat or Hilly area and (ii) The Non-Ghat or Plain country.

The formula adopted for the Hilly areas is $R = 0{\cdot}85\ P - 12$, where R is the average annual run-off in inches from the catchment and P is the corresponding average annual rainfall in inches over the total catchment area.

The formula used for Plain country is $R = P \times \left(\frac{P-7}{100}\right)$ the notations being the same as before.

Losses which appear as constant are taken into consideration in this formula. This formula can only be advantageously used in the Bombay State, as the area taken into consideration while deriving it is the area of the Bombay State.

(*b*) *Khosla Formula:* The formula is $R = P - \frac{t}{2} + C$ where R and P have the same notations as before, and t is the mean annual temperature in degrees of F on the entire catchment and C is a constant depending upon the factor that affects the surface run-off from the catchment area.

In his formula, Khosla has not only taken into consideration a constant C, but he has also considered t the temperature in the catchment area.

This is an improvement on Inglis formula, since temperature affects evaporation, transpiration, sunshine and wind also, which are the chief factors that generally affect the run-off. (It is learnt that this formula is being revised, to give better results, by taking into consideration the monthly temperature also).

Other formulæ in common use are.

(*c*) *Vermuele's Formula:* The formula is

$$R = P - (15{\cdot}5 + 0{\cdot}16\,P)\,(0{\cdot}05\,T - 1{\cdot}48)$$

where R is run-off, P is the rainfall in inches, and T is the mean annual temperature in degrees Farenheit.

(*d*) ***Justin's Formula :*** The formula is

$$R = 0{\cdot}9345 \times S\,\frac{0{\cdot}155P^2}{T}$$

Where R, P and T have the same notations as before and S is the slope of the drainage area, that is the maximum difference in elevation of the drainage area divided by the square root of the drainage area, i. e., $d / \sqrt{A}$.

(4) *Using Run-off Curves and Tables* :

(a) *Sir Alexander Binnie* is the earliest authority in estimating the run-off. His experiments were conducted on two rivers in Madhya Pradesh A table was prepared from the observations, of his experiments, which gave percentages of run-off from the rainfall in the locality.

The condition of the catchment, the geological, meteorological and other aspects which affect the run-off generally, are not taken into consideration by Binnie. The run-off given in his table is on the basis of the total rainfall in the catchment.

(b) *Strange* is the next authority on evolving the run-off. The results of his experiments are better than Binnie's. He conducted his experiments in the Bombay State. Strange also has not taken into consideration most of the conditions affecting the run-off.

In Strange's Table, the estimated run-off and the yield per sq. mile from the catchment area refer to the total monsoon rainfall only. Here the extent is divided into three classes. Good, Average and Bad Catchments, and the percentages of run-off to the rainfall are noted (for the total monsoon rainfall of the season).

Since Strange's Table does not differentiate between incidences of rain having the same total fall for the season, it cannot be used generally. For instance, the rain may be good for small period or it may be drizzle after a heavy rain for two or three days. A lot of variations is produced, from these, in the run-off.

Strange has prepared another table, to allow for these differences, in which the catchment area is divided into dry, damp and wet and the percentages of run-offs are given for daily rainfall in inches, and the run-off is computed from it. From this table, a chart has been prepared, vide fig. 14. When daily records of the rainfall are available, this method is better.

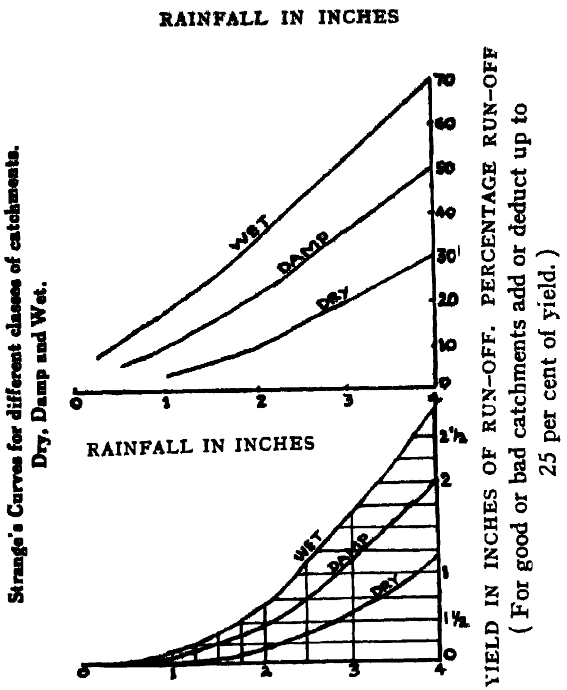


Fig. 14

(*c*) *Barlow's Tables*: The different classes of soils and countries and different falls of rain are taken into account by Barlow. He conducted his experiments in Uttara Pradesh, in catchments which were less than 50 sq. miles in each case; but the nature of the rainfall which is an important factor which affects the run-off is not taken into consideration by him. The total rainfall only is considered in calculating the run-off. On the basis of the daily rainfall also, Barlow has improved the same recently.

Note—Factors like the geographical, topographical, meteorological and surface conditions which affect the run-off are left out in the above said formulae. Even the small items assumed do not contain enough data for the purpose. Hence it is advised that each case should be studied independently and, as said above, the method of gauging is advised to be adopted.

15. The Catchment area for a storage reservoir, river weir or any irrigation work is the area of land which drains into the streams or river or reservoir.

Yield

16. Correct Information of Yield: The yield from a catchment should be correctly investigated, because, it should give an accurate idea of the quantity of water that can be impounded in the reservoir.

This yield should be sufficient for the purpose of supplying water to all the irrigable lands under the reservoir; also additional supply will be required in the reservoir, during the year of abnormally deficient rainfall; and during this year, the supply from the catchment is generally the least. These bad years occur at long intervals only, (as seen by experience) and the records pertaining to these years may not be easily available.

In water-supply schemes, the absolute minimum supply and also the minimum of two or three consecutive bad years, should be taken into consideration; but for an irrigation project it is enough, if only the average bad year's rainfall is taken into consideration.

Storage in a reservoir is generally expressed in million c. ft., and so, the yield is also sometimes — (only rarely) — expressed as million c. ft. per sq. mile of catchment area. The usual method is to express the percentage of the rainfall (i. e. the run-off) over the catchment area, as the yield.

17. Factors affecting the Yield: The factors, which affect the maximum discharge, also affect the yield and increase it, (and also vide para 4 under discharge) but there are some factors which act otherwise.

(1) *Pervious strata in the catchment area*: Here the water soaks into the ground, and is, for all outward purposes, seen to reduce the yield; but ultimately the water reaches the stream. This may not affect the yield very much. The pervious strata tend to reduce the maximum discharge and not the yield to the same extent.

(2) *Other factors*: like,

(a) *Gentle slope of the catchment area,*

(b) *Thick vegetation in the catchment area,*

(c) *The shape of the catchment area (long and narrow), and*

(*d*) *Reservoirs or marshes in the catchment area*: All these retard the run-off, and the yield is not much affected: but due to the retarded time, there is more wastage from evaporation.

18. Factors tending to Reduce the Yield into a Reservoir:

(1) Depression in the catchment area might retain a good proportion of the run-off water.

(2) When the cultivable land is cut in steps, it allows the ground to soak in more water than it otherwise does.

(3) Percolation and evaporation will be more, as the distance of the site of reservoir along the catchment increases (1/5 to 1/12 may be taken as the average run-off) and thus the yield is reduced.

(4) Percolation into the soil increases, if the land in the catchment area is cultivable, and the run-off is thereby reduced.

(5) Percolation due to sandy soil in the catchment area allows water to soak in (being gathered in underground streams).

19. Calculation of Yield from a Catchment: For flow irrigation, run-off is very important. Rainfall records are generally available and not the records of the discharge of the streams. The rainfall records also are not in some cases reliable, and so it is advisible to take actual gaugings of the stream, at least for a short period, and find the correct run-off instead of deriving the same.

Rainfall records, when they are available, should be got for at least 35 years, as the rainfall cycle in India is taken to be for a period of 35 years.

When a catchment area is of a large extent, with varying rainfalls, the area should be divided into sections, and the correct run-off assumed for each section from the amount of rainfall, studying other conditions also. To arrive at a more correct run-off, it is preferable to take run-offs per month, or even run-offs for the days of rainfall, and from these to take the total run-off.

The usual method of calculating the yield from a catchment is to take the mean annual rainfall of the area first, and from this the average bad year's rainfall is calculated. (This is taken as $\frac{2}{3}$ to $\frac{3}{4}$ of the mean annual rainfall).

The run-off is generally assumed or taken from Tables available.

The catchment area is obtained from the topo sheets with the help of a planimeter. Then the total available yield in c. ft. is arrived at.

Yield in c. ft. = Area of catchment in sq. ft. × Depth of run-off in it.

For example: If the catchment has an area of 2 sq. miles (or 2 × 28 million sq. ft.) rainfall is 60 in. and the coefficient of run-off is assumed as $\frac{3}{4}$, the yield is CA × rainfall × Run-off coefficient = (2 × 28) × 60 $\frac{3}{4}$ = 2,520 mil. c. ft.

If assumptions regarding run-off are not made as per Tables, empirical formulae (as already noted above,) for example formulae of Vernuele, Justin, Inglis, or Kutter may be used and the run-off and yield calculated.

Example: For the same question as above, if Inglis formula for hilly catchment is taken,

Run-off = 0·85 rainfall − 12 = 0·85 × 60 − 12 = 39

Yield = 2 × 28 × 39 = 2184 mil. c. ft. or 2200 m. c. ft. which is roughly equal to the above.

Discharge

1. **Discharge** is the quantity of water flowing at a cross section of a stream in a second.

The equation generally used for discharge is Q = Av

where Q is the discharge expressed in c. ft. per second or cusecs
A = area of the cross section of the stream in sq. ft.
v = velocity of the stream in feet per second

The velocity of the stream is found by the following methods:

1. Surface floats.
2. Velocity rods.
3. Current Metres.

N. B.—The mean velocity is taken as 0·8 times the surface velocity.

The formulae generally used to find v are

(a) Bazin's formula is $v = c\sqrt{rs}$

where c is a constant
r is the Hydraulic Mean Depth
and s is the slope

value of c is taken as $c = \sqrt{\dfrac{2g}{\mu}}$

$$\mu = \alpha\left(1 + \frac{\beta}{r}\right)$$

where α, β are quantities depending upon the nature of the soil. The values for α and β are given in the following table, for different soils.

	Soil	Value of	
		α	β
1.	Very smooth channels Ex.-Cement or planked surface	0 003	0·1
2.	Smooth channels Ex.-Ashlar or brickwork	0 004	0·2
3.	Rough channels Ex.-Rubble masonry	0 005	0·8
4.	Very rough channels Ex.-Earth	0·006	4·0

Ex. = Examples

Bazin's formula is used for artificial channels.

(b) Kutter's formula is also $v = c\sqrt{rs}$

where r and s are the same as above, but c is a constant and taken

$$c = \frac{a + \frac{l}{N} + \frac{m}{s}}{1 + \left(a + \frac{m}{s}\right)\frac{N}{\sqrt{r}}}$$

where $a = 41{\cdot}660476$

$l = 1{\cdot}8113250$

$m = 0{\cdot}0028075$

s and r same as above

and N = 0·20 for firm soil and gravel

= 0·25 for earth in perfect order free from stones and weeds

= 0·30 for earth in fair order with stones and weeds

= 0·35 for ground with much stone

Bazin's formula takes into account the general nature of the bed of the river and Kutter's formula takes into consideration the precise condition of the bed of the river, at the point of observation, and hence this is used for all streams big or small.

2. Maximum Flood Discharge: The maximum rate at which the water flows down the valley is called the maximum rate of run-off. This is expressed,

(*a*) in inches of water depth over the catchment in 24 hours, because the raingauge used in India is Symon's Raingauge, and the rainfall is measured once in 24 hours, or,

(*b*) in cusecs (if the volume as arrived at is divided by $24 \times 60 \times 60$.) For example, if

Q is the discharge in cusecs,

A is the area of catchment in square miles,

d is the depth of water of the run-off in 24 hours, then

A in sq. miles $= 8 \times 8 \times 660 \times 660 \times A$ sq. ft.

d in inches $= d\ 12$ ft.

To get the quantity in cusecs, in place of 24 hours, put $24 \times 60 \times 60$.

$$\text{Hence } Q = \frac{8 \times 8 \times 660 \times 660 \times A}{24 \times 60 \times 60} \times \frac{d}{12} = \frac{242}{9} \times A\,d$$

This is called the maximum or high flood discharge, or intensity of the maximum flood.

The maximum discharge from a stream or a river is produced when there is a rainfall of very great intensity, at a time when the conditions of the catchment area are favourable to a large run-off, (especially in humidity).

The maximum intensity of the corresponding flood discharge is absolutely necessary for the correct and safe design of all irrigation works, and specially for weirs, cross drainage works of the canal, flood embankments and the design of culverts and bridges.

3. Intensity Formulæ: There are different formulæ in use to find the intensity of the maximum rainfall. The following are some of the empirical formulæ in use:—

(1) ***Prof. Talbot's Formula,***

(*a*) For ordinary downpour of rain, $i = \dfrac{105}{t - 15}$

(*b*) For medium rainfall (which may occur only once in 15 years) $i = \dfrac{180}{t - 30}$

(*c*) For heavy downpours (which may occur only two or three times in a century) $i = \dfrac{360}{t - 30}$

where i is the intensity of rainfall in inches per hour during the period, t is the duration of rainfall of the intensity in hours.

(2) *Sherman's Formula.*

(*a*) For maximum rainfall, $i = \dfrac{(38 \cdot 64)}{t^{0 \cdot 687}}$

(*b*) For ordinary rainfall, $i = \dfrac{(25 \cdot 12)}{t^{\cdot 687}}$

4. Factors affecting the maximum discharge: The factors affecting the run-off enumerated in the last chapter will hold good for discharge also. In addition to them, other factors, which affect specially the maximum discharge of a stream, are noted below.

These factors may be either *natural* or *artificial.*

(a) Factors affecting the discharge under the *Natural Conditions.*

(1) *The extent, duration and intensity of rainfall*:—The intensity, specially in the case of small catchments, is very important.

(2) *The direction of the motion of the storms which carry rain*:—If the direction is downstream, it is expected that more floods will be got in the stream.

(3) *The size of the catchment area*: The maximum rate of discharge increases, as the size of the catchment area decreases. That is, for the same intensity the discharge is more in a small catchment.

(4) *Shape of the basin*: A fan shaped catchment produces more discharge than a fern or elongated shape.

(5) *Slope of the country*: Steep, impervious and deforested country produces more discharge, while a crooked country with small slope produces less discharge.

(6) *The storage in the catchment area*: This may be either artificial or natural. (A good storage takes away about

9 inches of rain). The storage extends the flood period and reduces the maximum rate of flow.

(7) *The amount of snow on the ground and also the temperature*: A good amount of snow melts with a high temperature and increases the flow in the stream.

(8) *The degree of saturation of ground*: If the surface and the soil below are damp and saturated, the discharge from the catchment is generally more.

(b) *Artificial Conditions*: To increase the flow under these conditions the following factors are helpful:

(1) Controlled storage.

(2) Deforestation.

(3) Reduced width of the stream.

(4) Construction of piers (for a bridge) which restrict the water-way and hold back the flow for a time and increase the flood.

(5) Formation of ice gorges.

(6) Failure of the dam or reservoir structure.

5. Methods of Finding the Maximum Flood Discharge: Maximum flood discharge can be found by any one of the following methods:

1. Rainfall Records.
2. Flood (H. F. L.) Marks.
3. Empirical Formulae.
4. Discharge curves.
5. Actual gauging.

6. Rainfall Records: These records are not easily available in full everywhere and it is generally difficult to get them for all the rain gauge stations in the catchment and for a number of years (say 35 years). If this information is managed, then the catchment area should be divided into small sections, with reference to the rainfall and topography, and the discharge from each section calculated and the total arrived at.

The catchment area is taken from the topo sheet, the rainfall from the available registers, and the maximum run-off assumed from records available, and thus the maximum discharge is got. (Vide Problems in the author's book–Solutions of Problems in Irrn. Engineering).

7. Finding the Maximum Discharge from Flood Marks: When reliable records of actual flood levels for long periods are available, they are the best guides to what may be expected in the future.

In the case of very large catchments, computation of maximum flood discharge from rainfall statistics is likely to result in very erroneous estimates. Hence it is imperative (in estimating maximum flood discharge) to rely more on flood levels.

In the absence of recorded flood levels, it is necessary to place dependence on the results of local enquiries regarding past floods, and these should be checked by reference to recorded rainfalls. Villagers will generally be able to point out, on the walls of houses or temples, or with respect to demarcation stones, etc., the limits to which their fields were submerged during floods.

The highest flood mark for any stream can thus be found by local enquiry. These flood marks should be got for a number of points by enquiring from different persons in the locality and checked with other verifications.

Having been satisfied about these marks (H. F. L.), the area of the cross-section of the stream at the point should be plotted and measured correctly. Then the longitudinal slope of the stream should be found out. By applying either Kutter's formula or Bazin's formula, the velocity can be found and the discharge at the site of the stream calculated.

8. Finding the Discharge by Empirical Formulae: The formulæ generally in use for finding the discharge are (1) *Dicken's formula,* (2) *Ryve's formula,* (3) *Fuller's formula.*

(1) *Dicken's Formula:* This formula is $Q = CM^{\frac{3}{4}}$ This was first devised for use in Northern India, but later on, by changing the constant, it has been adopted in all the states. The constant C varies from 800 to even 1600 according to the area of catchment and the amount of rainfall, but generally C is taken as 825, where the rainfall is between 25 and 60 inches. In Madhyapradesh, C is taken as 1000—1400 and in the Western Ghats as 1600.

(2) *Ryve's Formula:* $Q = C\,M^{2/3}$ where Q is the maximum flood discharge, C is a constant depending upon the nature of the

catchment and the intensity of recorded rainfall, and M is the area of the catchment in square miles.

Ryve's formula is applicable to South India. The constant C varies from 450 to 1000 or even more. It is generally used where the rain is less than 40 inches. In very flat country along the coast, C is taken as 450, and near hills as 675 and in between as 560. (For Madras area C is taken as 560).

Comparing Ryve's formula with the Dicken's formula, it is seen that in the former case, a much more rapid diminution of proportionate discharge is seen, as the area increases, than in the latter case.

Note: In estimating flood discharges, Ryve's or Dicken's formulæ should be used, only to check the calculations made, and no reliance should be placed on the discharges that these formulæ indicate, until their conditions of applicability have been settled on adequate and satisfactory data. The coefficient C to be used with each formula should be ascertained for the particular area.

In the Bombay State, *Inglis formula* is used generally for fan-shaped catchments. The formula is

$$Q = \frac{7000\,A}{\sqrt{A+4}} \text{ or } \frac{7000}{\sqrt{A}} \text{ approximately.}$$

where A is the catchment area in square miles.

(3) *W. E. Fuller's Formula*: This is also known as Flood Frequency formula. It is given as

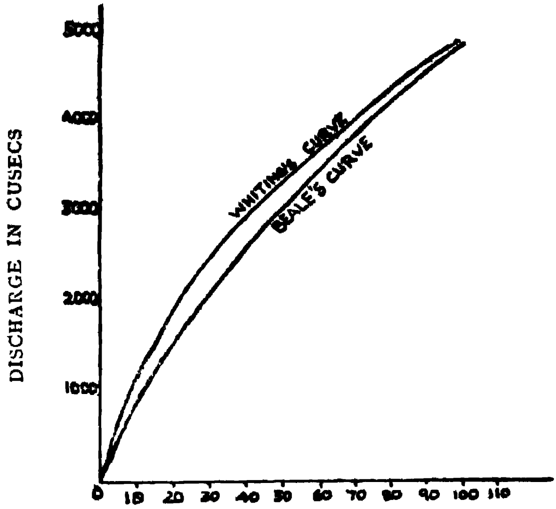
$Q = CA^{0 \cdot 8}(1 + 0 \cdot 8 \log T)(1 + 2A^{-0 \cdot 3})$ cusecs where C varies from 1·3 to 142 (average value 11 to 85).

T is the number of years after which such a high flood discharge is likely to recur.

9. Finding the Discharge by Discharge Curves or Rating Curves: The discharge curves used are generally of Beale and Whiting. These two curves are used in the Bombay State. These are plotted with the area of catchment as abscissa and the discharge as ordinate.

A graph showing the two flood curves is given below: Fig. 15. For rating curves, the curves are always drawn from previous experiments. When the high flood level is known, the discharge can be interpolated. The more the number of points on the curve,

and the nearer they are to the H. F. L. gauge, the better the result will be.



CATCHMENT AREA (IN SQ. MILES)

Fig. 15

10. Gauging: The correct discharges are obtained by actually gauging the stream and this is the best method to be adopted. If there is an anicut nearby, across the river, take the gauges above and below the anicut, find out the velocity of approach, and, knowing the length of the anicut, arrive at the flood discharge.

If there are no anicuts near by, set up a number of gauge poles at different points on the cross-section at the place, and find the maximum flood level and plot the cross section. Get the mean velocity and from this, find the discharge of the stream.

If there are reliable gauging records available, the maximum reading of the gauge is taken from each year's record for about 30 to 35 years, and then the discharge is calculated.

11. Flood: Floods have been occurring ever since the world began and man is fighting against them. In China the rivers Hoangho and Yangtse, and in India, the great rivers like Ganges, Brahmaputra and Indus, also the Mahanadi, Krishna, Godavari,

and Cauvery and even in Mysore State, the rivers Cauvery, Simsha, Kapini, Tunga and Vedavathi are doing havoc now and then. (In America, the river Mississippi is being trained for the last two centuries, but until now, no final success is gained by man against Nature.)

Floods are caused by general heavy rains of considerable dimensions, by a series of local storms and aerial cloud streams.

Flood discharges vary a great deal and depend upon

(1) Intensity and duration of rainfall,

(2) Size, shape, topography and geology of the catchment area,

(3) Extent and nature of forest growth,

(4) Climatic conditions of the country.

12. Factors Influencing Floods: (1) If the watershed is narrow and long, it has a less rate of flood run-off than if it were circular or fan-shaped.

(2) Basins with gentle slope act as detention-reservoirs, while steep slopes bring in floods.

(3) If the slope is gentle and the rainfall of low intensity, the flood is absorbed and if the soil is sandy and gravelly, the absorption is small.

(4) Surface vegetation affects the intensity of run-off and the flood.

(5) Existence of natural ways, ponds, etc., bring about greater uniformity of flow, and consequently there is reduction of peak floods.

(6) Deforestation tends to increase the rate of run-off and the flood.

(7) Floods are also caused by the encroachment on the natural water-way, (a) by structures built near towns and (b) by constructing embankment for road or railway line.

Questions.

1. Explain what is meant by run-off and mention the different factors affecting it.

(A. M. I. E. November. 1951).

2. Explain what is meant by "maximum" run-off from a catchment and mention the different factors affecting it. Give

the formulae to express the maximum rate of run-off from catchment areas of different sizes.

(A. M. I. E. November 1953).

3. Draw a neat sketch of a rain-gauge and detail out how it should be erected and maintained.

How are rainfall records used for the determination of (1) the total run-off from a catchment and (2) the maximum intensity of this run-off ?

(Mysore University, April 1954).

4. Mention the several factors influencing the maximum flood discharge from a catchment. Describe briefly the different methods of estimating the maximum flood discharge.

(Mysore University, March 1950).

5. What are the factors that affect the run-off from a catchment ? How is the average annual run-off determined ?

(Gujerat Univ. April 1955)

6. Distinguish between 'yield' from a catchment and the maximum flood run-off. On what do these depend, and how are these computed ? **(A. M. I. E. May 1951)**

7. What are the conditions which influence the relationship between the precipitation of rainfall in a catchment and the run-off ? If the rainfall data are available for a number of raingauge stations in a catchment, how would you proceed to find out the total annual precipitation in the catchment ?

(Bombay Univ. April 1949)

8. Write short note on : Flood Formula.

(Dept. of Technical Education, Bombay)
(Diploma in Civil Engineering, April 1954)

9. Describe how you would estimate the high flood discharge from a catchment for which rainfall data for a number of years are available, but the discharge observations do not cover the years in which very high flood occurred.

(Bombay Univ. October 1949)

10. What are the factors which influence the run-off from a catchment and what are the practical methods of computing it ?

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

11. What are the different methods of estimating the run-off from a catchment area ? Discuss in brief the merits and demerits of each method.

(Bombay Univ. B. E. Civil (New Exam.) November 1954)

12. Give at least four empirical formulæ and explain at least two rating curves you know for finding flood discharge from a catchment, indicating where each can be applied.

(Bombay Univ. B. E. Civil (New Exam.) Nov. 1954)

13. Discuss the factors influencing the maximum flood discharge from a catchment and different methods of estimating maximnm flood discharge.

(Bombay Univ. B. E. Civil (New Exam.) April 1955)

14. Given the rainfall figures of a number of stations in a catchment by what methods can you determine the average rainfall?

(Bombay Univ. B. E. Civil (New Exam.) Oct. 1955)

15. Enlist a few formulæ for evaluating the maximum flood discharge from a catchment. Which of them would you use for determining the maximum flood discharge at the site of a weir to be constructed in Mysore? Why would you not use the other formulæ ?

How would you determine the maximum discharge of floods from a given catchment apart from the formulæ ?

(Mys. Univ. B. E. Civil Sept. 1954)

16. Distinguish between the "yield" from a catchment and "the maximum run-off." On what do these depend and how are they computed ?

A reservoir has a catchment area of 75 sq. miles of which 45 sq. miles are intercepted by upper reservoirs. Calculate the maximum discharge assuming a coefficient of 450 in Ryve's formula. Design a suitable weir.

F. T. L. 50·00 M. W. L. 53·00.

(Mys. Univ. B. E. Civil April 1955)

17. Explain why, in spite of having the rainfall record of a number of years, it is preferable to actually measure the run-off from a catchment, even if it be for a single year, before launching upon an irrigation project.

Describe with sketches the best method of measuring this run-off.

(Mys. Univ. B. E. Civil April 1957)

(*a*) Distinguish between run-off and flood.

(*b*) Given the rainfall data for a catchment area, how will you estimate the yield and the maximum flood discharge ?

CHAPTER VI

APPLICATION OF WATER

1. Application of Water: For the proper application of water to the lands, the three items noted below should be carefully studied and followed.

A. On the lands to be irrigated one or several kinds of crops may be grown, and each one requires a different quantity of water; and it should be known beforehand, how much water is to be applied to the lands for each crop. This is called *duty of water.*

N. B. Provision should be made for any losses due of evaporation and percolation that usually occur.

B. Again it should also be ascertained how to use the available water economically, and to achieve this end, the *Phad system* and the *Block system* of irrigation have been developed.

C. When supplying water to the lands, as said above, a certain fixed quantity or calculated quantity is to be supplied, and for this purpose the method of measuring this quantity is to be settled. This measurement of water is done by what are called *Modules.*

D. Finally, knowing the quantity of water and the method of measuring this quantity accurately, one has to know further, how to apply this water to the lands-(the different plots of the land). These methods are *Surface irrigation* and *Sub-surface irrigation*, etc., which are further divided as *Border method, Basin method, Furrow method*, etc.

A. DUTY OF WATER

Definitions: Duty of water is the relation between the volume of water and the area of crop which it matures. It is defined as *the acreage which can be brought to maturity by the constant flow of one cubic foot of water per second, during the season the crop is standing on the ground.*

The base of duty is the time or period during which water is supplied to the crop. Base for rice is 160 days, and base for dry crop is 120 days.

Delta (Δ) *is the total depth of water required by a crop to come to maturity.*

3. Expression of Duty: Duty of water is generally expressed in one of the following ways :

(1) The most common practice is to *express the duty as the area that is served by unit volume of water.* That is, if one c. ft. per second of water, running continuously for four months, will mature 40 acres of crop, the duty (in this case) is said to be 40 acres to the cusec, to the base of four months, (Usually the base is not mentioned and only the duty is said to be 40).

(2) It is also expressed (in America) as *acre feet.* That is, the number of acre feet required to mature one acre of the crop. 1 acre × 1 ft. depth = (4840 × 9) × 1 c. ft. = 43,560 c. ft.

(3) Duty is sometimes *expressed in terms of the number of acres of irrigation which can be supplied per one million c. ft. of impounded water.*

(4) It is also *expressed in terms of the volume of water used per acre;* thus if it is said to be 200,000 c. ft. per acre, it means that, on the average, each volume of 200,000 c. ft. of water drawn from the tank is sufficient to mature one acre.

(Duty is also expressed as = 2 c. yds. per acre per hour or 0·015 cusec per acre, or 1 cusec = 66⅔ acres).

When a large amount of water is used on a small area of land, the duty is said to be low; and conversely, when a small amount of water is supplied to a large area of land, the duty is said to be high.

Formulæ: The following simple formulæ are used in connection with calculations of duty.

(1) $V = \frac{B}{D} \times 86400$

where, V is the volume of water in c. ft. used in maturing one acre of crop.

B is the base of duty, that is the number of days during which the supply of one cusec runs to mature the crop.

D is the duty of water, the number of acres matured by one cusec used continuously for the definite period.

(2) Delta $\Delta = \dfrac{86400 \times B}{43560 \times D}$ ft.

$$= 1{\cdot}98\frac{B}{D} \quad \text{or} \quad 2\frac{B}{D} \text{ ft. (approximately).}$$

$$= 24\frac{B}{D} \text{ inches.}$$

Delta is the total aggregate depth in feet (or in inches) if the volume of water is distributed over the whole irrigated area uniformly.

(3) $Q = \dfrac{A}{D} = \dfrac{A \times \Delta}{2B}$, where Q is the discharge in cusecs for the period B, and D is the duty and A is the acreage.

Example: In Mysore, for good quality of rice, the base is about $4\frac{1}{2}$ months or 135 days and depth of water or Δ allowed is 1 fathom (or 6 ft.), the duty is given by

$$D = \frac{2B}{\Delta} = \frac{2 \times 135}{6} = 45.$$

This is the normal duty allowed for paddy.

4. Seasons: The seasons for irrigation are generally divided into two periods: (1) The Kharif, (2) The Rabi.

(1) *The Kharif.* This season extends from 16th June to 15th October. Crops are sown in the beginning of the south-west monsoon and are harvested in autumn. The Kharif crops are rice, bajra, jawar and other food grains; also jute, tobacco, groundnut and other commercial crops. All these require more water than the crops grown in the Rabi season. (The quantity being 2 or 3 times as much).

(2) *The Rabi:* The crops in this season are sown in autumn and harvested in spring. It extends from 15th October to 15th February.

The usual crops in the Rabi season are wheat, barley, potato, gram, etc.

In South India, distinction cannot generally be made between the Rabi and Kharif seasons; because, most of the area in South India gets rain only from the north-east monsoon. Here the season is generally "hot for eight months and hotter for the remaining four months".

In Bengal and Madras, the Kharif crop consists mostly of rice. In Uttara-Pradesh and the Punjab, it is irrigated cotton and rice. In Madras, the whole crop is rice (in large canals, a high duty of about 60 is realised for the rice).

In the case of irrigation in the Rabi season, the demand for water is subject to great variations; but as the crops are also varying and are at different stages of maturity, simultaneous demand for water for all the crops is rare. The distribution of water is spread over a larger period than in the case of kharif crops. It is seen that the variations in Rabi duty are greater and more uncertain than those for the kharif. This indicates the impossibility of effecting economy in the use of water in widely scattered patches.

The lower duties are mainly due to large losses between the head of supply and the field. It is economical to irrigate by rotation, so that a certain length of the channel is in flow at one time so as to avoid waste, by sending down a continuous small supply through a large number of distributaries and field channels.

5. Natural Seasons: The monsoon season is the season for growing important food crops. This is different in different regions of India. For example, in the West Coast and in South India, the monsoon is for four months from July to September.

In Rajputana and round about it, the monsoon is only for two months in July and August.

In the remaining parts of the Union, the monsoon is from about 15th June to 15th October, (i. e. four months).

6. Crop Seasons (in different parts of India): (1) In Bombay (*Deccan*) the crop season is generally classed as noted below:

Name of Crop	Period	No. of days
Hot Weather Crop	15th Feb. to 15th June	120
Monsoon Kharif	15th June to 15th Oct.	122
Winter Rabi	15th Oct. to 15th Feb.	123
8 month's crops	13th June to 15th Feb.	245
Perennial Crop	All the year	365

(2) In the Punjab and Rajputana, there are only two seasons: (*a*) Kharif (*Katee-Katak*) (*b*) Rabi (*Chet*).

These names are given from the *month* in which the crop is usually harvested.

(3) In the remaining parts of India, the crop seasons are three in number: (*a*) Winter crop, (*b*) Hot weather crop, and (*c*) Monsoon crop.

7. Principal Crops in India, showing area irrigated:

Crop	Area in 1000 acres
Rice	58,110
Jowar	38,750
Wheat	24,500
Bajra	23,000
Gram	15,200
Cotton	11,350
Ragi	4,850
Sugarcane	3,200
Tea	730
Jute	600
Coffee	210

Note: Only important crops are mentioned here and the figures ~~are~~ for the year 1945–46.

8. Definitions of some important items, (which occur generally in the calculations for duty, etc.) are given below.

Capacity Factor: This is the mean supply of the canal divided by the full supply of the canal.

Crop Ratio: This is the ratio of the area under different crops of a particular channel to the entire projected area. This crop ratio is fixed in order to provide for the discharge in the canal, so that it may be uniform.

Cusec Day: (A cusec day on one acre roughly comes to 2 ft. depth of water for the day). One cusec of water flowing for one full day or 24 hours is called cusec-day.

Full Supply Coefficient: The full supply coefficient is the area estimated to be irrigated during the base period divided by the designed full-supply discharge of the channel at its head, during the maximum demand. This is also known as the *Duty on Capacity*.

Intensity: The number of acres, actually irrigated during the seasons, is first calculated and it is compared to the total irrigable area, and the percentage of this is taken as the intensity of irrigation. For example, if the total irrigable area is 5,000 acres and the area actually irrigated is 3,000 acres, then the *intensity* is

$$\frac{3000 \times 100}{5000} \text{ i. e. } 60 \text{ per cent.}$$

Nominal Duty: In Bombay, the cultivators have to apply for the water required for their lands to the Engineering Department, every season, in advance, and when their request is granted, they have to pay the full amount, irrespective of the area they irrigate. The duty assumed in this case is the *nominal duty*. It is the area for which the permit has been granted for the period divided by the mean supply for the base period.

Open Discharge: Open discharge is the ratio of the day cusec in the canal to the number of days the canal is allowed water for irrigation (and not the total number of days of the irrigation period). That is to say, it is the number of day-cusecs divided by the number of days the canal is in use.

Overlap Allowance: At the junction of two seasons, when water is switched over from one crop to another, the canal has to supply water for both the crops; and so it exceeds the normal discharge of the canal. The excess is generally called *Overlap allowance*.

The overlapping allowance is usually about 5 per cent over the usual kharif discharge. As said above, this excess is due to the overlapping of the crops.

Time Factor: The time factor of a canal is the ratio of the number of days the canal is in flow, to the number of days of the irrigation period. For instance, if in 10 days rotation period, the channel gets water for four days (day and night) and is closed for the remaining six days, then the time factor is 4/10; if the channel is open for 3 days (days and night) and for 2 days only during the day time, then the time factor is

$$\frac{(3) + (2 \times \frac{1}{2})}{10} = \frac{4}{10}$$

9. Duty under Reservoirs (or Tanks): For storage works in Madras 2,00,000 c. ft. or 4·59 acre ft. is taken for

the entire season, that is, 80 acres to the cusec for a base of 185 days.

In Mysore, one unit (or one acre × one fathom) that is, 261,360 c. ft. is generally stored for one acre of rice land.

In Madhya-Pradesh, the storage is taken at 80 acres for one cusec. (The storage provided is calculated at 16 to 22 acres per million c. ft.)

10. Importance of Duty Figures: (1) In an existing irrigation work the duty shows the efficiency of the management, by comparing the duties of similar works, and also by comparing it with the duty in previous years.

(2) For project works, the duty is a guide to arrive at the correct design of the channel for irrigation.

11. Working-out Duty: For working-out the correct duty under an irrigation work, the following information should first be gathered.

(1) The season.
(2) The crops.
(3) The system of growing the crop by irrigation.
(4) Number of waterings for the crop.
(5) The interval between them.
(6) The total depth for each watering.
(7) The total delta required.

For each of the crops grown under the channel or a tank, a crop-delta chart should be drawn, and the maximum or the highest quantity required should be taken for working out the duty. In this connection, it must not be forgotten to deduct for the rainfall during the season (average bad year's rain) for about 10 to 15 days.

12. Factors Affecting Duty of Water: The various factors that affect the duty of water are:

(a) Length of the season or the time taken for the crop to mature.
(b) Natural rainfall and evaporation.
(c) Soil conditions and canal conditions.
(d) Preparation of ground.

(e) Skill of irrigator.
(f) Crops raised.
(g) Care with which water is used.
(h) Cultivation.
(i) Chemical composition of irrigated water.
(j) Administrative practice.

(a) *Length of seasons*: If the season is short, the crops require generally less water as the loss will be small; and if the season is long, more wastage is done and more water is required.

(b) *Natural Rainfall and Evaporation*: Any rain occurring during the irrigation season takes the place of a certain amount of irrigation water that would otherwise be necessary. In *Malnads*, the natural rainfall is sufficient to mature the crops. In *Semi-Malnads*, the same is true in a lesser degree, but irrigation greatly increases the yield. In the maidans, the amount, time, and the manner of rainfall affect the quantity of irrigation required. The water evaporated from the soil is not available for the plants and varies widely with the climate.

The duty of water in storage works in the Bombay State is taken as follows (W. L. Strange).

November to February:	60 acres per cusec	Note the change with the seasons.
March to June:	40 ,, ,, ,,	
July to October:	80 ,, ,, ,,	

(c) *Soil Conditions* These may be taken as (1) Chemical and (2) Physical.

(1) The *Chemical Factors* that influence the amount of water used are

(i) The readily available soluble plant-food nutrients in the soil and an abundance of readily available plant-food compounds in the soil favour efficient use of water and heavy crop production.

(ii) The kinds and amounts of alkaline salts in the soil. Excess of alkaline salts in the soil requires copious (plenty) use of irrigation water, to leach the excess salt from the soil.

(2) *Physical Factors*: A loose sandy soil will not hold as much water as one of closer texture, and if the soil is also coarse, it may be difficult to apply sufficient water for plant needs, without heavy losses through the subsoil. Where the sub-drainage is very

free and the soil sandy and coarse, there is temptation on the part of the ryot to over-irrigate, and thus a large amount of water is allowed to waste, carrying with it much valuable plant-food and also injuring neighbouring lands.

Duties vary to a considerable extent with the nature of the soil irrigated. With the discharge calculated at the head of the distributaries as 10 days flow and 5 days closure, the duties are as follows:

Soil	Duty	
Sandy Soil	40 to 50 acres of rice	
Loamy	60	,,
Clayey	80	,,
	Maximum	Minimum
Retentive Soil	64	39
Medium soil	80	50
Porous soil	104	69

(d) *Preparation of Ground*: If the fields are not levelled properly, some lands will receive more water than required and others less; also if the different plots are not of uniform size, the plots will not receive water uniformly. Thus the manner of the preparation of the field will affect the economical use of water.

(e) *Skill of Irrigator*: The skill with which the existing water is used, also affects the duty of water. This varies with the experience of the ryot, or of the help he may employ. It is interesting to note that duties increase as irrigation develops, and with experience both on the part of the cultivators and on the part of Irrigation officers, it leads to improvement of duty. For example, for the Sone Canal in Bihar, the duty increased over the period 1885 to 1912 as shown below:

Year	1885	1886	1907	1912
Duty	47	54	72	73

(f) *Crops Raised*: Some crops require less water than others, and hence it should be seen that only the required amount of water is used, after experiment and experience.

(g) *Care with which water is used*: The factor that influences the use of water, concerns also those who are responsible for storage, conveyance and distribution of water.

Storage: A dam or reservior which stores water must be made as water-proof as practicable.

Conveyance: It is impracticable to deliver to the irrigators all the water taken from a reservoir or a river system. Even under the most favourable conditions, some water is lost in conveyance through leaky canal construction and by seepage and evaporation. Hence the canals should be made as water-tight as possible with lining, etc., wherever required.

Even with adequate facilities and skill, the water may be wasted by carelessness, if the necessity for care be not realised. Where water is abundant just as in the Visweswariah Canal in Mysore, or other river channels, its wasteful use is well seen. The best way to prevent such wastage is, to charge the ryots as per the quantity of water used, and not as per the area irrigated, as is the practice now.

Distribution: Proper alignment and section of channels should be designed, so that the ryots will get only the required quantity of water; also the distribution of water should be properly supervised and only the required quantity of water allowed to the ryots. Most of the large canals have spillways or waste-weirs that carry back to the river, part of the water. These spillways are intended to protect the canal when the quantity of water in the canal is suddenly increased by rains, etc.

(h) *Cultivation*. If the surface of the soil is kept loosened to a considerable depth and the fields are kept clear of weeds, etc. which would consume a great deal of water, a much higher duty can be realised. On virgin lands the duty is lower than on lands which have been previously irrigated.

(i) *Composition of Irrigation Water*: Excessive amount of the alkaline salts in the irrigation water sometimes necessitates relatively liberal use of water as means of assuring adequate conveyance of the harmful soluble alkaline salts through the soil, and away into the drainage water. If the water contains fertilizing matter (manure) the duty is more.

(j) *Administrative Practice*: (1) Delivery of water to irrigators in continuous flow and small streams generally encourages wastefulness and decreases duty.

(2) Rotation or turn method of delivery encourages careful use of water.

(3) Delivery on demand, that is, if the irrigator requests for the water and then gets it, it stimulates the irrigator to use water very carefully.

(4) Method of payment for water, whether by rate for the amount of water used, or per acre of irrigated land. The former method gives more duty, as the cultivator uses the water more economically.

(k) *The condition of the canal*: If the soil (through which the canal is taken) is of coarse structure, there is more loss by percolation, and if the soil is heavy (containing clay) it is good for preventing percolation.

The canal should be given the maximum slope, so that the flow may be rapid. This increases the duty under the channel.

The irrigated land under the channel should be concentrated and not dispersed here and there. In the former case, the duty is naturally more.

(l) *Time and Frequency of Tilling*: If the cultivation of the land has been for a long period, the land would become saturated and the duty is naturally more.

If frequent tilling is done, it reduces the loss of moisture through the weeds, and also evaporation from the soil, where ground water is within the capillary reach of the surface.

(m) *Configuration of the Land*: Flat and shallow channels give smaller duty than steeper and deeper ones.

13. Duty Influences the Size of the Canal to a great extent: The dimensions of the main canal are primarily determined by the duty of water during the period of maximum pressure; that is, the greatest quantity that may be required during a short time, when the greatest demand depends upon the crop and the distance.

14 Methods of Taking Discharge to Determine the Duty under a Canal: Discharges for ascertaining duty are calculated in two ways.

(1) The actual discharge at the head of the canal (near a regulator) is measured. This is called the discharge at the head or *Gross discharge.*

(2) From the above figure, losses due to (a) the water wastage, (b) run-out in the escapes, (c) used in navigation, and (d) other losses such as evaporation, absorption, etc., are deducted. The result is called the *net discharge* or the discharge actually utilised. This is a more correct measurement of the quantity of water which it is necessary to draw for irrigation.

15. Variation of Duty with Rain: The duty is proportional to the rate of percolation from the soil. Even in retentive soils, it is greater due to the period (4 to 5 months).

16. Duty under Canals: The duty under canals is generally not uniform throughout its length, because during conveyance of water, there is loss due to evaporation, percolation and absorption; as such, the muasurements for duty are taken at three points noted below:

(1) At the main head sluice: This is called *gross quantity.*

(2) At the entrance to the branch channel: This is called *lateral quantity.*

(3) At the head of the land to be irrigated: This is called *net quantity.*

In any particular channel, these duties increase as the channel flows to the tail end, for example, the duties for a channel were found as follows:

(a) The duty at the canal head was 53.

(b) The duty as utilised in general was 72.

(c) The duty at the discharge outlet was 104.

In Bengal, the duty at the head of a canal was calculated to be 90 on the average, and actual discharge used was 125.

In the Punjab, the duty at the head was an average of 115 and for the whole year it was 150.

The losses between the head of the main canal and the distributary outlet are estimated to be roughly 20 to 40 per cent. The Royal Commission reports that in British India the wastage in canals is from 30 to 50 per cent.

17. Duty Assumed at the Head of the Distributaries for Different Seasons and Crops in the Bombay State:

Crop	Duty
Hot Weather Crop	75
Monsoon (Kharif)	200
Rabi Dry	110
Eight Months, Crop	120
Perennial: Sugarcane	50
Vegetables	60
Rice	50
Cotton	90
Wheat	120
Jowar	125
Bajra	200
Gram	150
Groundnut	135
Fodder	150
Lucerne	75

The duties for projects generally assumed at the head of a distributary, in the Bombay State are on a sliding scale, as noted below:

Command (area) in acres	Up to 1000 acres	1000 to 2000	2000 to 3000	3000 to 4000	4000 to 5000	5000 to 10,000	10,000 to 15,000	15,000 to 20,000	20,000 to 30.000
Duty (Assumed)	80	80 to 100	100 to 110	110 to 120	120 to 130	130 to 145	145 to 160	160 to 175	175 to 200

18. Duty in Madras: Under tanks in the Madras State, when the soil is loamy and retentive, a duty of 60 is generally assumed for a base of 140 days.

The duty under delta area is assumed as 80 to 110.

In the regions where the river and the country are steep and the irrigated areas are porous, the duty is between 40 to 50.

N. B.—The duty under the inundation canals for dry crops is 60 in Sind, and 70 in the Punjab and for rice it varies from 30 to 40.

19. Duty under Well Irrigation: It is generally more than in the case of tank or canal irrigation. The reasons are

(1) Water is more economically used.

(2) The canal from a well, being of short length, and lined throughout, the losses on account of evaporation and absorption and the loss due to any breach in the canal embankment are all reduced to a good extent.

(3) The distribution work in well irrigation is done according to the will of the cultivator and his actual requirements.

20. Duty in Different Seasons:

State	Duty arrived at during	
	Kharif or monsoon season	Rabi or Winter season
Bengal	65	70
Uttar Pradesh	55	160
The Punjab	80	115
Bombay	100	115

21. Duty Usually Assumed in All Projects:

Crop	Duty
Rice	40
Sugarcane	50—60
Cotton	80
Dry Crop	80—120
Monsoon Dry Crop	160
8 Months' Garden Crop	60
Perennial Crop	60

22. Duties as Realised in Different States:

State	Crops	Duty
Bombay	Nira Left Bank Canal	
	Perennial Crops mostly	
	Sugarcane	60
	Eight Months' Crop	80 to 100
	Four Months' Crop (dry)	120 to 150

Madras	Rice	66
The Punjab	Lower Chenab Canal	
	Wheat	270
	Upper Bari-do-ab Canal	
	Wheat	200

In the recent project, in the Cauvery Delta, the duty for rice is between 50 and 60.

For Kharif crops in the Punjab and Uttar Pradesh, the duty is 100 and for rabi crops, it is 200 and 250.

In Madras, the duty for rice in delta areas is 80 to 110, and the duty in porous soils where the area irrigated is steep, long and narrow, it is between 40 and 50.

Duties vary (as stated elsewhere) with the climate and rainfall of the locality. The following statement gives a clear idea of the same.

State	Rainfall in inches	Crop	Duty
Bengal	60"-80"	Rice	90
		Wheat	180
		Sugarcane	120-180
Orisa	55"	Cotton	250
		Sugarcane	65
		Wheat	200
		Rice	100
Madras	15"-20"	Rice	60
(Tungabhadra		Cotton	180
Project)		(Hot Weather)	
Bombay	30"	Rice	70
		Cotton	210
		Wheat	100-250
		Jowar	210
Sind	5"	Rice	50
		Wheat (rabi)	200
		Cotton (Kharif) dry)	100

23. Improvement of Duty: To improve the duty under an irrigation work, the following measures will have to be adopted:

(1) Use the irrigation water as quickly as possible, to avoid wastage in evaporation and percolation.

(2) The length of the canal should be the minimum possible.

(3) The irrigation water should be confined to channels only, especially field channels or *hikkals*, instead of allowing the water to flow from land to land.

(4) Avoid alignment of the channel or canal in either sandy soil or fissured rock.

(5) Design the velocity in the channel to be the maximum, so that the soil of the channel can stand it and the water may go safely and as rapidly as possible.

(6) Allow the maximum quantity of water to the irrigated land, so that there may not be wastage by allowing small quantities often.

(7) Arrange for lining the channel wherever it is suspected that there may be absorption or percolation or seepage.

(8) Introduce the system of rotation of water, or the block system, or the rotation of crop.

(9) Redistribute the irrigated land, so that the ryots may get only as much land as they are capable of managing.

(10) Introduce the system of levying water rate as per the quantity of water used by the ryot.

(11) Whenever possible, introduce or construct parallel channels and pick-up weirs.

(12) Train the staff of the channel, both the officers and the men (*Nirgantis*).

(13) Demonstrate to the ryots the economy of water, through the Government department.

(14) Introduce the Irrigation Act and see that it is properly enforced.

(15) Establish research stations in the locality to study the soil, the seed, and conservations of moisture.

24. Watering Dry Crops : The most economical condition for watering dry crops is when water is applied simultaneously to an equal depth throughout the field. The area of each plot of the field to store water should be made as small as possible, by providing a small embankment on the lower side, and at the same time the rate of flow into each of these areas should be sufficiently large to prevent wastage of water.

If the system of irrigation is through wells, these areas are specially small, (ranging from 500 to 1000 sq. ft.). In Northern India these small areas are known by the name *kiaris*.

The first watering is generally more than the subsequent waterings. The first is about 4 in. and the subsequent ones 2 in. to 3 in.

In Madras, in the Cuddapah-Kurnool Canal, the first watering allowed usually is 6 in. and the subsequent ones about 4 in. once in about every 20 days. The main points observed in supplying water to the dry crops here are that the distributive channels are made to run full during supply, and that on other days the channel is completely closed. Thus a duty of about 120 is got under this canal.

In Northern India, for the Rabi crops, wheat and barley, about 4 or 5 waterings are allowed for a base period of about five to six months, and the duties range from 110 to even 220. The kharif duties for the same dry crops range from 60 to 120.

25. Waterings for Rice : For rice, water is required not only for the actual supply to the crop but also for the preparation of the land itself. To begin with, special seed beds are made for the rice in about 3 to 5 per cent of the area to be transplanted and the seedlings are grown here, and then they are transplanted into the main fields prepared for it.

26. Preparation of Land : For the preparation of the land required for paddy cultivation, it is flooded with water, and then this land is ploughed while in this state, so that it may be thoroughly cultivated and the surface soil properly puddled. Water is kept in the field for some days, with a view to killing any of the plants or weeds. When the seedlings in the bed are ready, the surface of the field is drained off its water and transplanting begins. The operation of preparing the field takes about a week or ten days, and a good quantity of water (say about even 12 to 20 inches total depth) is required on the field.

In some places there is no transplantation at all, only the seeds are sown by *broadcasting* them over the field. This is done where the rainfall is heavy, ranging from about 50 to 60 inches and where advantage is taken of the natural rainfall. For example in Malnad areas and the West Coast.

After transplantation, water is allowed to each of the plots once in four to eight days, so as to submerge the lands until the crops come to maturity.

27. Water Required for Rice Cultivation: Preparation takes 8 to 10 days. Depth of water required is 12 to 24 inches.

After transplantation, water is required for about 110 days, and the total depth is 44 inches. For maturing the crop, it takes generally about 20 days additional, and no water is generally required during this period.

28. Summing up the above, it may be said that the yield from a soil generally increases, if the supply of manure and water is given to it up to a certain limit. But many of the ryots, when an irrigation scheme is freshly executed, allow more water than required and then they realise that the yield is not as they anticipated. That is to say, the yield decreases after a certain limit of the supply of water and manure.

For any crop, the first watering should generally be more (say about 4 inches) and the subsequent watering less (say about 2 to 3 inches). If a particular crop requires more water, the same should be allowed, but it should be done in a number of waterings and not by increasing the depth of water suddenly over the field.

Delta or depth of water required. The usual depth of water (or delta) allowed on different crops is given in the Table below:

Name of Crop	Total depth of watering required in inches
Rice	45—72
Sugarcane	48—60
Tobacco	30
Cotton	20
Ordinary Cereal	12

In all these cases, deduction should be made for any rainfall during the period.

The delta allowed in the Punjab is:

Crop	Delta
Wheat	0·8 ft.
Sugarcane	1·8 ft.
Vegetables	1·2 ft.

29. Delta and the Number of Waterings for Some Crops :

Name of Crop	Number of waterings	Depth per watering in inches	Total depth in inches
Wheat	7	1·5	10·5
Indian Corn	5	2	10·0
Cotton	5 to 6	About 3 to 4	20
Sugarcane	20 to 39	2 to 2·5	40 to 60
Gram	1	3 to 4	3 to 4

B. ECONOMICAL USE OF WATER

1. Rotation of Crops: Definition: By rotation of crops, it is meant the change in the nature of the crop grown in successive years on the same land, which sometimes is allowed to lie fallow.

Necessity for Rotation: Rotation is necessary to secure restoration of fertility to the soil, which, without it and without artificial help from manures, becomes exhausted if constantly irrigated by tank water, as such water is practically devoid of the fertilising silt which canal waters usually carry. The rotation should be as high as possible, because the revenue chiefly depends upon the perennial crop of the whole area under irrigation. The cultivation of the same crop year after year in the same soil is termed *mono-culture.* In the early days of agriculture, only mono-culture was adopted; but later on, it was found that soil got tired speedily, if put under the same crop continuously; but if different crops were raised, the soil could withstand it better.

3. Advantages of Rotation of Crops: (a) Though all crops need the same kind of plant nutrients, all of them do not take it in the same quantities or proportions; some crops favour certain plant nutrients and take them more than the others. Thus, if a particular crop is grown, year after year, the soil gets deficient in the plant food favoured by that crop, which gets exhausted and becomes the limiting factor of growth. If different crops were to be raised, there would certainly be more balanced feeding.

(b) Again, crop diseases and insect pests will multiply at an alarming rate, if the same crop is to be grown continuously. Rotation will check this disease.

(c) A leguminous crop, if introduced within a particular rotation, will increase the soil nitrogen, and thus enhance the fertility potential.

(d) As the crops within the system of rotation will have among them, both shallow rooted (cereals) and deep rooted ones (crops like cotton, tobacco, etc.) they will feed both from the upper and the lower layers of the soil, and thus the soil will be better exploited.

(e) As rotation is varied farming, including cash crops, food and fodder crops and soil renovating crops, it will, in the long run, benefit both the farmer and the soil in more than one way.

4. Phad System of Irrigation: Phad or Thal system of irrigation was the first standard system of irrigation that came into existence in the Bombay State. This system has been in use on some of the Bandharas of Nasik and Khandesh Districts.

Under this system of irrigation, the land to be irrigated is divided into three or four equal parts called the Phads or Thals. Hedge lines demark these Phads. Again, each Phad is subdivided into a number of plots according to the number of irrigators or owners in the village. Once again, the individual plots are separated from the others by boundary stones. In a year only one crop is grown in one Phad. In different Phads, different crops are grown. That is, if in Phad No. 1 Rice is grown then in Phad No. 2, sugarcane will be grown and so on. The corps in the Phads are interchanged or changed in the subsequent years. In the Phad system of irrigation, crops like Rice, Sugarcane, Rabi crops and Kharif crops are usually grown. Each crop will be grown in all the Phads in three or four years. This rotation of crop will continue without break, except during the fallow period, which is an interval between different crops.

One or two officials (selected from among the villagers) are employed to look after each Phad or Thal. A man known as Neerganti (as called in the Mysore state) or Patkari (in Bombay) is appointed for the purpose of releasing water from the irrigation

channel to the individual plots, according to the schedule fixed in consultation with the villagers.

Under this system of irrigation, the demand of revenue or assessment is uniform, for according to the available water and the measurement of crops (annually), the areas in which different crops are grown are fixed. This assessment is also called Composite or Consolidated Assessment. Land revenue and water tax are included in this assessment. Whether the land is actually irrigated or not, the irrigator has to pay this assessment. This system ensures water supply to all the commanded and irrigable lands. The necessary rest to the soil is got by proper rotation of crops. Theoretically all have equal losses, under this system of irrigation, in a bad year, though actually the big land lords get more than their share.

5. Block System of Irrigation : The object of the system is to distribute the benefits of irrigation works, over a large number of villages and to concentrate the irrigation in each village, within blocks of specified limits and in selected situations. The total area of the block in each village should be large enough to enable every one, who is able to grow an irrigated crop, to have a share, but not too large to constitute a surfeit, or lead the cultivators to neglect the advantages of water-supply in good seasons.

Usually the area is divided into three portions, of which one portion will be supplied with water sufficient for perennial crops, the second one for semi-dry and the remainder, only for dry crops. For the permanency of the irrigation, rotation of crops is very essential. But after a heavy crop, the land requires rest.

Introduction of Block System in the Deccan Canal Tracts : *The main causes which led to the introduction of the Block system of irrigation in the Deccan canal tracts are*

(a) Scanty or sparse population.

(b) Limited means of ryots.

(c) Want of manurial properties in the water.

(d) Malaria engendered by the clear water irrigation.

(e) Other diseases produced by waterlogging and over-saturation.

(f) Salt efflorescence brought about by introducing an abundant supply in a dry and thirsty tract.

For the above reasons the return from the reservoir falls far short of expectation, either from want of keen demand for water, or on account of sickness.

6. Selection of Blocks: The following points should be considered while selecting the blocks :—

(a) The area allowed to each block should measure not less than 50 acres and only one third of the area in each block should be grown with wet or perennial crops and the remaining two-thirds, with semi-dry and dry crops respectively.

(b) The blocks should be located in places, which the canal water can command, regard being had to the quality of the soil, proximity to the canal and actual area irrigated in the locality in the year past.

(c) The combined area of blocks for each village should not be more than one-third of the cultivable area under command.

(d) Water should be given to each block, from one canal outlet only (as far as possible) and the quantity of water supplied to the land by measurements, this quantity being determined with due consideration to the nature of the soil and the crops grown.

(e) No block or portion of block should fall within a quarter mile from the village proper.

7. Facilities Created by Government to Improve the Block System :

Co-operative Bank : Allied institutions like Co-operative Directorate, aided by the State and under the supervision of the Government, will go a great way towards helping the ryots. Multi-purpose Co-operative Societies are recently established by Government in almost all the villages. Chief among the other advantages will be the rapid development of the land and the cultivation of high class crops with imported seeds and improved methods.

Agricultural Farm: An agricultural farm on a moderate standard and improved methods of cultivation and manufacture will also be a great advantage. The farm can be made almost self-supporting and Government should supervise the affairs of the farm.

Introduction of the Irrigation Act: For working the Block system successfully, it is necessary that the Irrigation Act (otherwise called the Canal and Drainage Act) be introduced and enforced.

8. Block System of Irrigation under the Visveswariah Canal (Mysore).

Objects of the System: (1) Under this system as the ryot or the cultivator under the canal is allowed to grow three different kinds of crops, the duty realised is great, and a large tract of the country is brought under irrigation, without affecting the health of the people. It benefits the largest number of people in the area, where there was once precarious rainfall.

(2) As only one-third of the irrigated land is under paddy at a time, there is not much water-logging and unhealthiness.

(3) The rotation of crops introduced in the system is beneficial to the ryot and brings more income to him. If paddy is grown for three years, a ryot gets $3 \times 80 = 240$ Rs., but as per block system he gets

First Year	Paddy	Rs.	80
Second Year	Dry Crops	Rs.	53
Third Year	Sugarcane	Rs.	500
	Total	Rs.	630

That is, the ryot will get more than 2½ times the value got, if paddy alone is grown.

(4) The system of rotation of crops, unlike the growing of paddy which takes only 5 to 6 months, will employ the ryot for the full year and in addition, there is economical use of water.

9. Essential Features: (1) The land is divided into small blocks, or units varying from 45 to 450 acres. They are subdivided into *Hanthas* or plots, 15 to 50 acres and three *Hanthas* form one unit, and each *hantha* is provided with a field channel.

(2) *Rotation of Crops*: The crops are rotated once in three years in the order of D–S–P, i. e., Dry crop in the first year, Semi-dry or Sugarcane in the second year and Paddy in the third year.

(3) There is no restriction on the kind of crop to be grown. The ryot or the cultivator can grow any crop he likes, only water is rationed as per the block system, i. e.

(a) If a dry crop is grown, water is allowed to the crops once in 20 to 30 days during the crop period of about three months only.

(b) If semi-dry or sugarcane is grown, water is allowed to the land, once in a week or ten days and during this period, the water will be '*on*' for 2 or 3 days and '*off*' for the remaining days; of course, water will be allowed for the complete period of 10 to 16 months (or even 18 months sometimes) until the sugarcane is cut.

(c) If paddy is grown on the land, water is continuously allowed from 10th June to 10th January, until the paddy is fully matured.

Note: It need not be said here that, for preparing the land and for the sowing of the seeds, the required quantity of water is allowed in time.

10. Difficulties Experienced by the Ryots and Remedies Suggested: (1) There were complaints from the ryots that, as the crops changed every year, the land had to be prepared specially for the particular crop. From experience gained, it was seen that it was only a matter of small expenditure, and it would pay them to do the work.

(2) Some ryots complained that the manurial properties of the soil were lost, while changing from one crop to another. This in fact is not so, because each individual crop has got an affinity towards only some particular chemical constituents of the manure.

(3) There were complaints that due to the new system, there was over-production and the prices became reduced. In this connection, it may be said that the ryot is at liberty to grow any kind of crop. (Only the system of water supply is restricted) and if one crop does not pay, he can grow any other which pays him better.

(4) There were complaints that as the ryot cannot grow paddy every year, which is his staple food, the system is a handicap. The complaint is only frivolous, as any kind of crop can be easily got from the neighbouring ryot by exchange.

General: The block system is intended to provide the maximum benefit to the largest number of people, and also the greatest increase of wealth to the country. Again due to competition, good economical and commercial crops can be newly introduced in the state, and thus the general prosperity of the country developed.

11. Advantages of the Block System of Irrigation:

(1) The land will not be overwatered or injured by water logging or salt efflorescence.

(2) Proper rotation of crops will be maintained.

(3) As the loss of water in transit is minimised, there will be much economy in the water supply.

(4) The Government staff economises the supply and thus the number of water canals required is small.

(5) As the area under irrigation is in blocks, the cart tracks are not obstructed.

(6) Insanitary conditions and diseases due to over-saturation will be removed.

(7) On account of the economy of water under the system, a larger area will be irrigated than is possible at present and the total produce and prosperity of the tract will be increased to the mutual advantage of the cultivators as well as the Government.

C. MODULES.

Introduction: In India, in most of the places, water given to the ryots for irrigation is charged according to the area irrigated. The ryot did not mind the quantity of water used, and there was much wastage; also the supply depended upon the difference of the head of water (between the parent channel and the field channel) and whenever levels allowed, the cultivator would deepen his field channel and draw a larger supply (to the lower lands) leaving little or none for higher lands which needed the water most.

When there was difficulty of water, *the rotation system* or doling out water by "turns" was introduced, just to manage with the existing insufficient supply.

To overcome the above defects, (in some places) the system of distribution of water by volume is introduced. The supply in the channel will be varying due to many causes:

(1) The supply at the head of the distributary varies during regulations.

(2) Within the distributary (a) when the outlets are closed, or (b) when irrigators misuse water.

(3) With the condition of the channels (as the sandy bed shifts and as the cultivators put obstructions across the channel to [illegible] water through the outlets to their lands).

Hence, for the varying heads between the two channels, some arrangement to give a uniform supply is necessary, and this is effected by a Module or Gauge-outlet.

Irrigation outlets are of 2 types *(a)* Non-Modular and *(b)* Modular.

A. Non-Modular outlets: An outlet where the discharge is dependent on both the upstream and downstream water levels is called a non-modular outlet.

This type is used in all old channels, and even now where lift irrigation is in good usage, there is less loss of head in this type.

B. Modular outlets: A module is an outlet or device which ensures fixed supply (within certain limits) independent of the water surface levels of the channel into which the supply is delivered.

Modules are classified as (1) Perfect or Rigid module and (2) Semi-module or Flexible module.

(1) *A Rigid or Perfect module* is a device for ensuring a constant discharge of water, passing from one channel to another, irrespective of the water level in each within certain specified limits.

This is used where the ryots are charged according to the quantity of water supplied to their lands.

A rigid module may have *(a)* moving parts, *(b)* non-moving parts. Examples of the latter type are (i) Foote Module, (ii) Spanish Module and (iii) Gibb's Module.

(2) *Semi or Flexible Module* is a device the discharge through which varies according to the head of water in the parent channel, but is unaffected by the downstream fluctuations. The device works as if it had a free fall; honce the discharge varies with the upstream level. It is used mostly in irrigation channels, because when there is excess supply in the channel, all the outlets take proportionately higher discharges, and where the level in the channel is low, all the modules will draw proportionately decreased discharges, as otherwise there will be less supply at the tailend.

Examples of this are, Kennedy's Sill Outlet, (ii) Kennedy's Gauge Outlet, (iii) Scratchley's Semi-module, (iv) Harvey-Stoddart's Semi-module and (v) Crump's Semi module.

Proportional Module: A semi-module is said to be proportional when it draws off its supply in direct proportion to the discharge flowing in the parent channel. Here the flexibility is unity.

Flexibility is the ratio between the rate of change of discharge of the outlet to the rate of change of discharge in the main channel.

Rateable Outlet: When an outlet can be arranged to be set, so that it will give a fixed discharge (under given conditions) it is called a Rateable outlet.

Modular range: Modular range is the range with various factors, which a module or a semi-module works; or it is the range between two limits (the upper and lower limits) of any factors beyond which the outlet cannot act as a module.

Depression head: This is the depth of a point below water level in a module, in terms of which the discharge is expressed.

In Europe, the system in vogue is always supply of water to the cultivator according to the volume of water used by him. The devices employed at the beginning were (1) Italian module and (2) Spanish module.

(1) *Italian Module*: Here the sluice is regulated by the hand, and an approximately uniform head on the notch in the chamber is arranged to be maintained.

The system involves constant supervision and manipulation by the hand, and is neither practicable nor automatic, and so it is imperfect and not much used.

Spanish Module: Here the area of the orifice is made to vary as the head of the water changes. A conical brass plug attached to a float is allowed to work in a circular opening or orifice made in the floor of a masonry chamber. This plug works vertically in guides. The water from the main channel enters the chamber, passes through the circular opening in the floor into a well below and then makes its way into the distributing channel outside, below the bank of the channel (Vide Fig. 16).

SPANISH MODULE

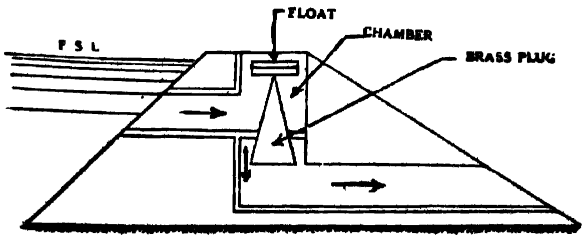


Fig. 16

1. Formula for Discharge: The discharge through the orifice is given by the formula

$$Q = CAV = C(r^2 - x^2) V = C(r^2 - x^2)\sqrt{2gh},$$

where, Q is the discharge in c. ft. per second,

C is a constant,

r is the radius of the orifice,

x is the radius of the plug,

h is the depth of discharge, and

g is the acceleration due to gravity.

For different value of h, x can be found out, and the plug designed accordingly. The main objection here is the great fall required which is not easily available in all cases.

2. Requisites of a Perfect Module: A module, if it could be made perfect, would have to meet the following requirements.

(1) Steady automatic discharge of the volume allotted to the water course, notwithstanding variations of the head, within fixed limits.

(2) Small loss of head.

(3) Freedom from discharge by silt or weeds.

(4) Portability.

(5) Absence of complicated mechanism.

(6) Provision, so far as is reasonably practicable, against derangement but without outside interference.

(7) Means by which the outlet could be opened or closed by the cultivator.

(8) Indication of working head or discharge.

(9) Increase of discharge when the level of the water in the distributary rises above a certain height.

(10) Absence of any obstructions to the free passages of a reasonable share of silt from the distributary into the water-course.

(11) Cheapness and durability.

3. Kennedy's Rateable Gauge Outlet: Kennedy's rateable gauge outlet pattern B satisfies all the above requirements of a perfect module except (2) and (9).

There is a bell-mouthed entry from the distributary channel, and it meets a vertical bent pipe, which extends above the high flood level of the distributary channel at the narrowest point of the mouth piece. This vent pipe has air holes at the top, and under normal conditions, contains no water. Beyond the junction of the two, there is a long expanding cone, extending under the bank to the outside, into the field channel (Vide Fig. 17).

KENNEDY'S GAUGE OUTLET.

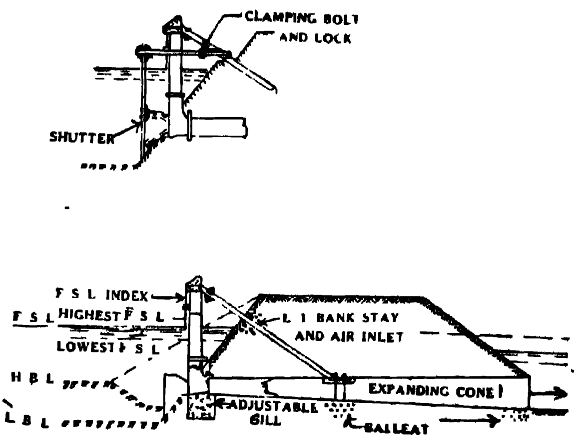


Fig. 17

The water entering the bell mouth from the distributary, shoots across the air in the vent pipe, and enters the small end of the long cone. Here the velocity naturally decreases and the pressure increases (as per Bernoulli's law) until the water forces its way outside into the field channel.

4. Kennedy has made the orifice semi-module a practical possibility: The velocity of flow at the orifice is converted into static head.

Kennedy's device is something like a venturi tube with free admission of air to the. throat, so that the throat pressure cannot fall below zero, (as long as the total head is sufficient to bring the throat pressure to zero the pressure is fixed at zero and the discharge depends on the supply level).

It is a semi-module and eliminates entirely the effect of the level of water in the field channel; the only condition being that h_0 or the depression head (i. e., the head from channel F. S. L. to the centre of the orifice) should not be greater than $5h$ or 5 times the difference of head in the two channels. The effect of this is that *the control of supply to the field channel lies entirely in the hands of the authorities,* whereas with the old pipe outlets, the cultivator could increase the supply by deepening the channel (by increasing the working head).

As the discharge is determined by depression head only, a scale can be drawn on the vent pipe, so that the number of cusecs passing at any time can be seen at once.

(The loss of friction here is only $0{\cdot}21\ h_0$, and h_0 should not be more than $4\frac{1}{2}\ h_1$ practically).

Note: Modules are not absolutely necessary on other channels except field watercourses; but in the former case also, they are used, as they serve for the regulation of water which is effected now and then by manual control (as per requirements of these channels) and can be used as meters.

There are *other types of modules* in use, viz,

(1) *Gibb's modules in the Punjab.*

(2) *Wilkin's modules in U. P.*

Wilkin's module is a metal appliance and is easily portable.

5. Gibb's Module: Gibb's module is a device for obtaining a constant discharge, even though the head in the parent channel (the distributary) is varying up to a certain limit. Gibb's module is of rigid type and has no moving parts, (Vide Fig. 18).

It consits of

(1) An inlet pipe connecting the parent channel or the distributary to a rising pipe. The inlet pipe has a controlling shutter at the upstream side.

(2) A rising pipe taking a turn or spiralling through 180° and entering an eddy chamber.

(3) An eddy chamber having the floor horizontal and semi-circular in plan, and discharging into an open spout. Vertical baffle plates are introduced in the chamber along its circular length

at equal distances, leaving an annular space below the chamber. The lower edge of the baffle plates is inclined from the inner wall to the outer wall of the chamber.

GIBB'S MODULE

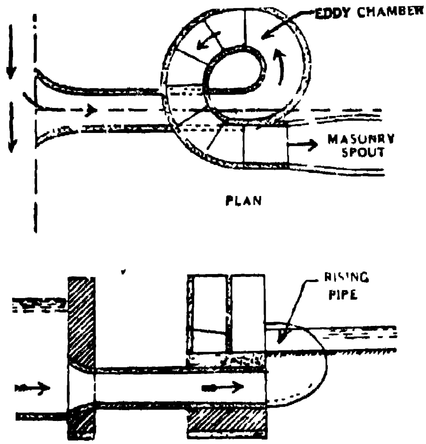


Fig. 18

(+) A masonry spout having a divergent masonry flume on its downstream side.

Water from the parent channel or the distributary gets into the rising pipe through the inlet pipe. When the water flows into the chamber through the rising pipe, it attains a free vortex motion. (In free vortex motion, the product of velocity and radius is constant). In this condition, the water, at the outer circumference of the stream, flows at a comparatively high level due to the centrifugal force.

In the eddy chamber, the water gets its velocity reduced, when it dashes against the baffle plates, and gets a rational flow, on account of the blades provided in the bottom of the eddy chamber. Until the extra head is dissipated successfully, the water struggles in the different chambers provided for the purpose. Afterwards the water discharges into the watercourse through the divergent

flume. A divergence of 1 in 10 is given to the spout to increase the downstream watercourse.

The discharge through the module is given by the formula

$$Q = r_o \sqrt{2g}\,(d_o + h_o)^{3/2}$$
$$\left\{ \frac{m^2 - 1}{m^3} \log_e m + \frac{1}{m} \log_e m + \frac{m^2 - 1}{2m^3} \right\}$$

where, Q = discharge,

r_o = outer radius of chamber,

d_o = depth of water near the periphery of the eddy current, or the level of water at the outer wall of the chamber,

h_o = height of water or the head of water causing the flow,

m = module ratio, i. e. the ratio of the outer radius to the inner radius of the chamber.

The discharge through the module is proportional to the discharge in the parent channel up to a certain limit. After this limit, even though the head varies, the discharge is constant or stationary up to another limit, and again if the head is increased further, the discharge again varies.

It is economical to construct the module in reinforced cement concrete when the discharge is from 1 to 3 cusecs; and when the discharge is more than 3 cusecs, it may be built of brick masonry.

The difference between the minimum and maximum heads over the module within which the discharge is constant is called the "*modular range*". This modular range can be found by drawing a curve of the discharge for various heads. The modular range depends upon the perfectness of the maintenance of the vortex motion of water in the module.

The modular range can be increased by increasing

(a) The number of baffle plates provided in the chamber (The minimum number of baffle plates generally provided is 6).

(b) The length of the rising pipe.

(c) The number of volutes.

(d) The number of eddy chambers.

(e) The length of the exit flume, (when the length of the exit flume is $2d$, the modular range is maximum).

Disadvantages of Gibb's Module. (1) It is a masonry structure; and it takes much longer to put it in place and it cannot be carried from one place to another. (2) It cannot act when the water falls below a certain level. (3) It draws silt from the distributary and so the working is affected. (4) It is costly.

For other types of Modules, see authors' book on Solutions to Problems on Irrigation Engineering.

6. Standing Wave. According to Barnoulli, when water flows, there is a particular condition of flow called the *Critical condition of flow*. It may be mathematically defined as that condition of flow when the change of velocity head is just enough to produce a change in the elevation of water surface. The velocity and depth are also referred to as critical velocity and critical depth in this condition of flow.

Critical velocity V_c is given by $V_c = \sqrt{D_c g}$ where D_c is the critical depth. Velocity head in critical conditon of flow is

$$k = V_c^2/2g = D_c/2.$$

If the depth at which a stream is flowing in an open channel is less than V_c^2/g and if the depth is increased by an obstruction, then at the point where $D = V_c^2/g$ (critical depth), the water surface tends to become normal to the bed and a standing wave is formed.

A standing wave may be defined as one which persistently forms at the same place. As stated above, this happens in an open channel when the flow is between the critical and neutral depth, and the neutral depth is positive and greater than the critical depth. This condition may occur downstream of irrigation falls, when the depth available downstream is not enough to form a hydraulic jump. Standing waves can be noticed either above or at the foot of a weir, or on the downstream side of bridges discharging in flood.

7. Hydraulic Jump: Hydraulic jump is a phenomenon produced when a shallow stream moving with a high velocity strikes water of sufficient depth. It is the abrupt jump of water in the region of impact between the rapidly moving stream and the more slowly moving wall of water. There is a great rumbling of the coming water and much foam is produced throughout the moving mass.

The conditions required for the formation of a Hydraulic jump are

(i) The flow upstream of the jump should be less than the depth below the critical condition of flow.

(ii) The flow downstream of the jump should be greater than the depth above the critical condition of flow.

(iii) The neutral depth should be positive.

The formation of hydraulic jump is quite distinct from that of standing waves. However, the term standing wave is applied to both (especially in the Punjab Irrigation practice). Waves are frequently mistaken for a hydraulic jump, because of the splash of water accompanying them.

If the mass of water *abcd*, of high velocity V_1 and small depth D_1 (Vide Fig. 19) such that V_1 is greater than $\sqrt{g D_1}$, impinges

HYDRAULIC JUMP

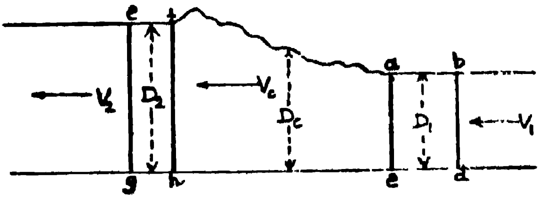


Fig. 19

against a mass *cfgh* of depth D_2, such that velocity V_2 does not exceed $\sqrt{gD_2}$, then somewhere between these masses $V_C = \sqrt{gD_C}$. (V_C and D_C are critical velocity and depth respectively). Hydraulic jump occurs across the critical depth.

For a hydraulic jump on level floor, we have

(1) The depth of the stream leaving the standing wave

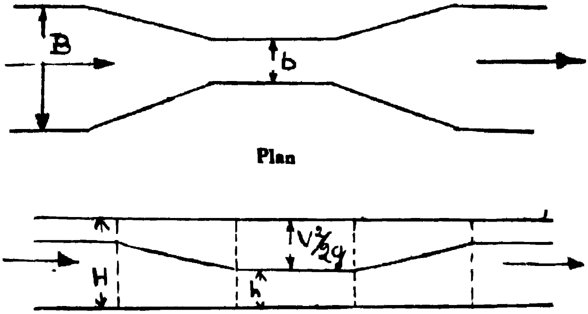
$$= D_2 = \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}} - \frac{D_1}{2}$$

(2) Height of jump $= D_2 - D_1$.

(3) Loss of energy $= \dfrac{(D_2 - D_1)^3}{4D_1 D_2}$

8. Venturi Flume: The venturi flume is a device to measure the quantity of water flowing in the channel. Here the

normal width of the channel is reduced, i. e., it is made smaller at the flume. (See Fig. 20). The quantity of discharge in the flume is given by the equation,



Section

Venturi Flume.

Fig. 20

$$Q = \frac{BH \times bh\,2g}{B^2H^2 - b^2h^2} \times H - h \quad \text{or} \quad = \frac{Aa\,2g}{A^2 - a^2} \times H - h$$

where, Q is the quantity of discharge in cusecs,
B is the normal width of the channel,
H is the height before entry into the flume,
b is the reduced width in the flume portion,
h is the height in the flume portion,
A is the area in the normal section, and
a is the area in the flume section.

This equation does not provide for any loss due to friction.

The *difference between a standing wave and a venturi flume* is as follows:

A standing wave is used when the head in the channel is sufficient to use it as a semi-module, the discharge here is a function of the upstream depth only.

Whereas, a Venturi flume can be used, even when the head in the channel is not sufficient to use it as a semi-module; also the discharge varies with the difference in the water level upstream and the water level in the throat.

D. METHODS OF APPLICATION OF WATER TO LAND

Methods of Irrigation: The two main methods of applying irrigation water to agricultural lands are (1) Surface Irrigation and (2) Subsurface Irrigation. Of these the Surface Irrigation is used everywhere (all over the world).

Under the Surface Irrigation, three main categories are included namely (A) Flooding Method, (B) Furrow Method and (C) Spraying Method.

Under the Subsurface Irrigation, (a) Natural Sub-Irrigation and (b) Artificial Sub Irrigation are the two methods.

(A) Surface Irrigation: *Flooding Method.* This is further divided into (a) Free flooding method (b) Border method, (c) Check method, (d) Basin method and (e) Surface pipe method.

(a) *Free Flooding or Wild Flooding* In this method, water is distributed to the lands from the field ditches from which it flows, on account of the natural slope of the land, without any embankment to guide or prevent it. (Vide Fig. 21). Water will

FREE FLOODING METHOD SHOWING FIELD DITCHES

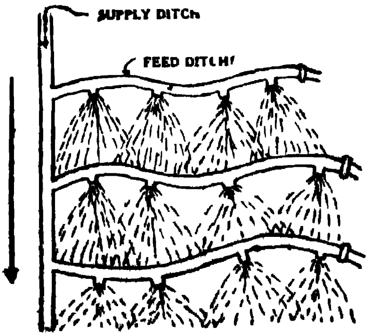


Fig. 21.

thus cover all the cultivated land, while flowing from one ditch to the other, these ditches are constructed at all controlling stations and properly spaced. In this method, the water cannot be controlled properly. Sometimes, check gates are used on supply

ditches permanently. For field ditches, canvas, steel or earthen dams are built, to check the flow of water. The cost of preparing the land for this type of irrigation is low.

The main disadvantage of this method is the small stream and the large labour cost. This is best adopted and largely used in areas where the period of the crop is small and about 2–3 crops are grown per person.

(b) *Border Method* : In this method, the land to be irrigated is divided into long narrow strips extending lengthwise along the natural slope and each is separated by two borders parallel to each other. This confines the water which flows in a sheet-like form within these strips. The water is supplied from supply ditches and wets the soil as it advances. This type is a modification of the free flooding method and the flooding here is guided by field embankments.

In this type, larger streams can be handled and a large field can be irrigated without excessive loss due to percolation. The cost of preparing the land is greater than that of free flooding, but the cost of distributing water to the lands is comparatively less This method is also called "*Strip checks*," "*Ribbon checks*" and "*Gravity checks*" according to their shapes, to distinguish them from contour checks.

BORDER METHOD

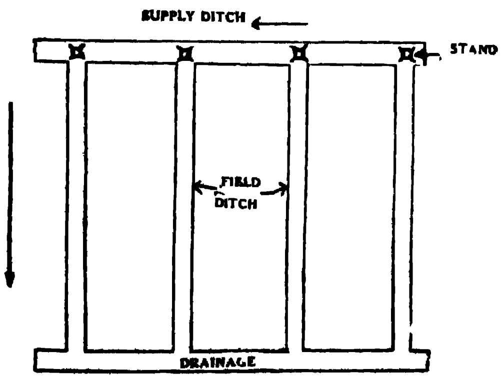


Fig. 22.

This method is best suited to a soil, of rather wide variation in texture, one which absorbs water readily, so that the depth required for the particular crop is absorbed from the water as it flows.

Permanent structure of wood or concrete are built in each ditch to control the flow of water and to deliver it. Flow into the check is usually stopped before the water reaches the lower end, the water flowing in the ditch being sufficient to complete the irrigation. Proper drainage is provided to remove the waste water at the other end. (Vide Fig. 22.)

(c) *Check Method*: In this method, the land to be irrigated is level and is surrounded by embankments in which the water supplied may stand until the water required is absorbed.

Check methods are of two types. They are (i) *Rectangular check* and (ii) *Contour check.* These are distinguished by their shapes.

In rectangular checks, the land is rectangular in shape and is at right angles to the sides of the field and it is connected by embankments or levees at proper places (Vide Fig. 23).

In contour checks the land is irregular in shapes and levees are provided along contours having proper vertical intervals and these are connected by cross levees at suitable places. (Vide Fig. 24).

RECTANGULAR CHECK

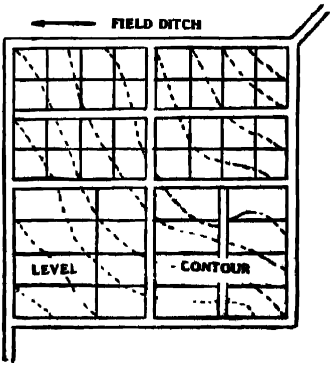


Fig. 23.

These may be used for flooded crops. They are also used for Furrow corps like cotton, sugarcane, etc. These are best suited

CONTOUR CHECK

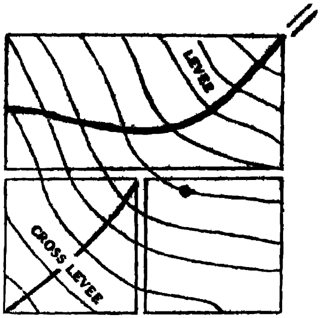


Fig. 24.

to heavy soil, where water is to be stagnant for several hours, so that the land may absorb the required water. They are also suited to very permeable soils which must be quickly covered with water in order to prevent excessive losses near supply ditches through deep percolation.

The cost of preparing the land for these types varies with the extent of cut on the high side and fill on the lower side. For lands of small and regular gradient, the cost may be as for Border method on the same land; for irregular directioned or flat land, the cost may be even less. Contour checks are cheaper than rectangular checks.

(d) Basin Method: The method is an application of the checks to the orchards. The checks are made small, i. e. a basin is constructed for each tree, and as such there is no necessity for levelling the land within each basin. Each basin is rectangular in shape and contains one tree in it: recently contour checks containing groups of trees are used (Vide Fig. 25).

This method is used in orchards, when the furrow method is not very suitable, i. e., in heavy soils, where the wetting of the partial areas in the furrows makes it difficult to secure sufficient absorption, and in the porous soils where excess of percolation occurs with limited cross capillary movement, at the upper end of the furrows. Since the full storage capacity desired by the soil is obtained in the basin method, it is used in winter irrigation also.

BASIN METHOD

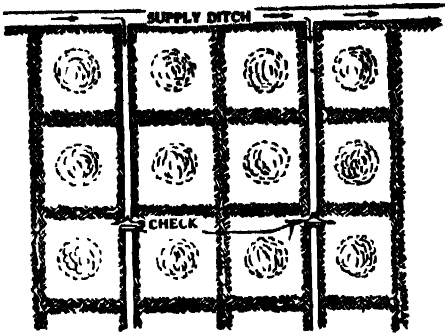


Fig. 25.

In this method, water is supplied to the lands either directly or run from basin to basin, but the former method is better. The cost of preparing the land is small.

(*e*) *Surface-pipe Method*: In this method, water is applied to the land through slip-joint metal pipes, placed on the surface of the land. Water is conveyed from permanent pipes to the surface pipes.

Uses: This method is largely used where the soil is hard and heavy grading is not allowed by the subsoil conditions and where only small streams are available. This method is an improvement of the flooding method, and the surface pipes are used instead of field ditches. Since water can be sent to any higher region under pressure, there is not much work in preparing the land, and therefore the cost is low.

Sprinkler Irrigation. Vide Fig. 26. It is well suited to irrigating coarse, sandy and gravelly soils, especially when these soils are shallow and cannot be prepared correctly for surface irrigation. It can also be used quite successfully on steep and irregular slopes.

Advantages: 1. Crops can have water, (a) when they need it, (b) where they need it and (c) in the controlled quantities that they need.

2. It eliminates surface run-off and deep percolation losses, thus reducing the wastage of the irrigation water and conserving the water supply.

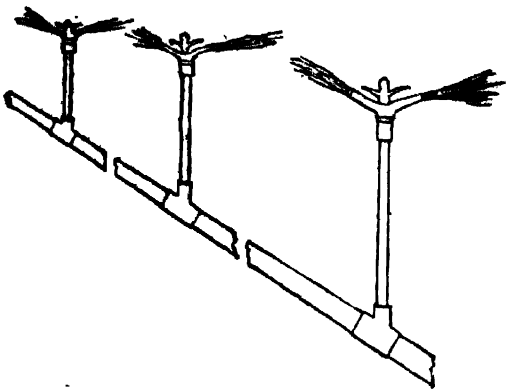


Fig. 26.

Aluminium Irrigator Sprinkler.

3. It applies water without run-off and thus eliminates soil erosion.

4. As there is no need for field ditches, (a) maintenance is considerably reduced and (b) the land available for cropping is increased.

5. No particular skill is required to uncouple, move and recouple the light aluminium laterals used commonly for the works.

6. The entire farm can be irrigated from the first year, as ditching and land levelling of new farms are not necessary.

Recently Aluminium Irrigation pipes are being used to a very great extent in America, Australia and elsewhere. It is seen that about 20,000 miles (weighing about 40,000 tons) were manufactured in the year 1955. This system is said to give 200% increase in field crops, such as, wheat and oats, and about 400% increase in forage crops.

Details:—Water is forced by a pipe, through a main line to which light weight portable pipe sections are fixed by couplings. These side lines or *laterals* are provided with sprinkler heads, to produce the required spray. These sprinklers are provided at intervals, so that the sections sprayed may overlap. As soon as

one section is sprayed, the connections here are removed and fixed to the next section and the work of spraying continued to the full area.

Messrs. James Wright Ltd., Calcutta, are dealing in the equipment (required for the sprinkler system) manufactured by Messrs. Ame's Irrigation Pty. Ltd. The following is an extract from their pamphlets, which show the uses and advantages of the system.

Perfo. Rain Sprinkler System

Effective irrigation is of fundamental importance to farming activities. The ability to spread the right amount of water quickly, easily and economically, is often the difference between profit or loss. Properly controlled irrigation results in superior crops, higher yields and lower operating costs.

Effective irrigation is *planned irrigation.* There are two popular methods of portable overhead irrigation: Revolving Sprinkler Systems and *Perf-o-rain, Roto-rain.* Revolving Sprinklers apply water in overlapping circular patterns under relatively high pressures. *Perf-o-rain,* operating under very low pressures, spreads a uniform 'gentle rain' over a rectangular area through a pattern of holes along the entire length of the pipe.

Perf-o-rain is the simplest, most efficient, 'rain making' system ever developed. It sprinkles water gently and evenly over a rectangular area. Any ordinary pumping system or gravity flow from elevated dams, ditches, creeks or tanks, can serve perf-o-rain through permanent or protable mains. Perf-o-rain pipe sizes are usually 2" to 6". Sand or gravel do not clog the holes. It covers large or small acreages with ease and speed, assuring bigger and better crops. Low in initial cost and capable of operating at minimum expense, these systems pay for themselves in 2 or 3 seasons, sometimes in a single season.

Advantages: 1. Complete, uniform penetration of the root areas.

2. Efficient distribution means tremendous water savings.

3. No water loss from run-off on normally porous soils.

4. Eliminates labour and tools required for building and maintaining ditches, furrows and basins.

5. Space saved provides more land for cultivation.

6. Readily portable by one man.

7. When pipe is moved, land is left free of obstructions for fast and easy cultivation.

8. No land levelling costs or loss of natural fertility from moving top soil.

9. Soil erosion reduced to a minimum.

10. Promotes fertility conservation.

11. Water is spread so gently that the rich top soil does not wash away.

12. No over penetration.

13. Leaching and washing of natural or applied fertilizers is stopped.

14. Harmful alkalis do not rise to the surface.

Lo-Head Gated Pipe Controlled Furrow Irrigation

Furrow irrigation is accomplished by running small streams of water by gravity between the evenly spaced rows of crops planted on accurately sloped or levelled land. Effective furrow irrigation depends on efficient use of all available water and consistently careful control of the water flow in each furrow.

Advantages: 1. It replaces inefficient open ditch systems for carrying water to the furrows.

2. It eliminates troublesome head gates and laborious ditch bank cutting.

3. In addition, it provides easy, precise control of water volume in each furrow, assuring the most uniform water coverage possible.

4. Light weight and readily portable, a small amount of AME'S LO-HEAD GATED PIPE serves a maximum acreage with minimum use of labour.

5. It is the most economical and efficient method of controlled furrow irrigation.

B. Furrow Method: In this method, to obtain enough soil wetting, water is conveyed to the lands through small ditches or furrows. This is used largely for crops grown in rows, like onion, etc. or for orchards and cereal crops. In the case of some soils and crop requirements, the soil can be wetted to only about 1/2 to 1/5 of the surface area using furrows for irrigation. This reduces

surface evaporation, lessening the puddling of heavy soil and it also helps to cultivate the land soon after the irrigation is over.

Furrows are of two types: (i) Corrugation furrows and (ii) Contour Furrows.

Corrugation Furrow Method: It is the method in which

CORRUGATION FURROW

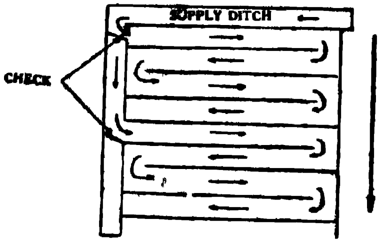


Fig. 27.

the furrows are used for cereal and forage. This is used on soils where erosion occurs on account of flooding. This is an application of the furrow method to crops where furrows cannot be used after irrigation. This method is specially useful, where the available stream is small.

The furrows in corrugation are more shallow than contour furrows. The water is distributed to the furrows either through earth ditches, flumes or pipes (concrete or other structure). (See Fig. 27).

Contour Furrow Method: This is a combination of checks and furrows. The land is divided into strips with proper slopes and furrows by which they are irrigated and trees are planted in rows, parallel to the contour (See Fig. 28).

CONTOUR FURROW

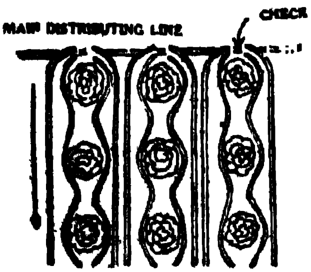


Fig. 28.

This method is largely used in orchards where the land is steep. The cost of preparing the land is small and the conveyance of water to the lands is safe. Water is distributed to the lands either by flumes or pipes.

(C) Spraying method: This method is also called *sprinkling method.* It is very costly and so it is used only for high value crops.

Water is applied to the soil in the form of a fine spray, like ordinary rain. This method is better than the other methods, in distributing water uniformly to smaller depth of irrigation. Losses due to percolation and surface evaporation can be altogether eliminated by this method.

Sub-Surface Irrigation: This is also called *Sub-Irrigation.* This is divided into two methods: (a) Natural sub-irrigation and (b) Artificial sub-irrigation.

(*a*) *Natural Sub-Irrigation*: In some places, the natural onditions are favourable for the supply of water to soils directly under the surface and this is known as " Natural Sub-Irrigation".

This is suitable to impervious subsoil at a depth of 6 ft. or more, a porous loam or sandy loam surface soil, uniform topographic condition and moderate slopes. The control of water-table on the lands which have a high water-table to prevent waterlogging, etc., results in economical use of water, rich harvest and low labour cost.

As this method prevents surface evaporation, soil-baking and surface distribution, attempts are made to popularise it.

(*b*) *Artificial Sub-Irrigation :* To get high value crops in small areas, under suitable conditions, water is conveyed to the soils through pipes or conduits, laid well below the surface of the soil, and this is called Artificial Sub-surface Irrigation.

This method can be employed successfully in soils which allow free lateral movement of the water and relatively high capillary movement. The cost of this method is rather prohibitive, and therefore it is not much in use.

Questions

1. What is block system? What led to its introduction in the Deccan canal tracts? What are its advantages?

(Bombay University 1949)
(Poona University, Nov. 1953)

2. Write a short note on: Phad system of Irrigation.

(Bombay University, April 1950)
(Poona University, October 1952)

3. Write a short note on: Block System of Irrigation.

(Mysore University, April 1954)
(April 1949, March 1947)

4. What are modules and semi-modules? Why and where are they provided? Illustrate your answer with suitable sketches.

(Poona University, April 1955)

5. What is a Hydraulic Jump? What are its practical applications? Derive the formula for the height of the jump.

(Gujarat University, 1955)

6. Write a short note on: Standing Wave.

(A. M. I. E., November 1952)
(Poona University, April 1954)
(Mysore University, March 1945)

7. What is an Irrigation outlet? Distinguish between

(a) Modular and non-modular outlet.

(b) Flexible and rigid modular outlets.

What is meant by adjustable proportional semi-module? What is its importance?

(Gujarat University, 1954)

8. What is the difference between a semi-module and a rigid module when used as irrigation outlets?

(A. M. I. E., November 1951 and 1953)
(Gujarat University, November 1953)

9. Distinguish between venturi-flume and standing wave flume. **(Gujarat University, 1953)**

10. Explain with sketches the working of Gibb's Module.

(Bombay University, 1949)
(Poona University, November 1953)

11. Write a short note on semi-module.

(Poona University, October 1952)

12. Write a short note on Modules.

(Mysore University, March 1948)

13. Name the various methods of applying irrigation water on the agricultural land and describe briefly, with sketches, any two methods.

(Gujerat University April 1953)

14. Distinguish between Duty and Delta.

(Gujerat University, April 1955, 1953)

15. Define 'duty' of water and discuss the several ways in which it is expressed.

Differentiate between 'Duty inclusive of rainfall' and 'Duty exclusive of rainfall.'

(Mysore University, April 1954)

16. (a) Define the terms 'duty', 'delta' and 'base' as applied to irrigation in India, and how are they expressed?

(b) Enumerate the facts which affect 'duty' of crops.

(c) Give an equation connecting these notations for expressing 'duty'. **(A. M. I. E., May 1953)**

17. Define the terms 'duty' and 'delta' and give the duties of principal crops in the Bombay State. Explain why the duty varies on different canals, and also on the same canal at different places.

(Gujarat University, November 1953)

18. Explain the term: Duty

(A. M. I. E., November 1951 and 1953)

19. What is meant by the "duty of water" and on what does it depend?

A small reservoir with a capacity of 1,250 acre feet is able to irrigate with two fillings an extent of 500 acres. In another system, a channel carrying 60 cusecs is able to irrigate 3,000 acres. The irrigation season is 140 days in both the cases. Which system is using water economically?

(A. M. I. E., May 1951)

20. What is meant by the duty of water in irrigation? How is it expressed? What measures would you suggest to improve duty in an existing canal system? Give figures of duty for the following crops:

Paddy, Jowar, Wheat, Cotton and Sugarcane.

(Mysore University March 1951 and April 1950).

21. What is meant by duty of water in irrigation? How is it expressed?

What is the base for paddy?

(a) What are the duties generally assumed for paddy, sugarcane, and semi-dry crops for irrigation projects in the Mysore State?

(b) How would you improve the duty of an old river channel?

(c) Why are duties low in the Mysore State as compared with deltaic areas?

(Mysore Univ. Sept. 1955)

22. Distinguish between (1) modular and non-modular outlets and (2) a semimodule and a rigid module.

(Bom. Univ. B. E. Civil (New Exam.) Nov. 1954)

23. What is meant by proportionality and adjustability of a module?

(Bom. Univ. B. E. Civil (New Exam.) Nov. 1954)

24. Draw a dimensional plan and section (not to scale) of a Kirkpatrick's orifice type module. Explain its working in brief.

(Bom. Univ. B. E. Civil (New Exam.) Nov. 1954)

25. What is meant by flexibility, sensitivity, and proportionality of a module? Explain why the setting for proportional moduling in the case of an orifice-type module is different from that of an open type module.

(Bom. Univ. B. E. Civil (New Exam.) April 1955)

26. What are the usual types of measuring devices used for canals? What do you understand by a proportionate module?

(Bom. Univ. B. E. Civil (New Exam.) Oct. 1955)

27. Illustrate the method of arriving at the discharge at the head of a distributary with the help of assumed areas of mixed crops. What is time factor and where is it used?

(Bom. Univ. B. E. Civil April 1956)

28. Write explanatory notes on the Block System.

(Bom. Univ. B. E. Civil April 1956)

29. Explain the terms Setting and proportional moduling. Why is setting for proportional moduling different for different types of modules.

(Bom. Univ. B. E. Civil. April 1956)

30. Give the advantages and disadvantages of a rigid module and a semi-module.

(Bom. Univ. B. E. Civil Oct. 1956)

31. What do you understand by the terms duty and delta? Give the average duties of principal crops in the Bombay State.

Why does duty vary on different canals and also on the same canal at different places ?

(Gujerat University S. E. Civil April 1956.)

32. Write explanatory notes on Rotation of crops.

(Gujerat University S. E. Civil April 1956.)

33. What is meant by duty of water in irrigation ? Give figures of duty for sugarcane, paddy, wheat and cotton. What are the measures adopted to improve the duty in a large irrigation project ?

Find the capacity of a reservoir which has an atchkat of 7500 acres of which 2500 acres is under sugarcane which requires water for 250 days in the year, 3500 acres is under paddy requiring water for 160 days and 1500 acres under dry crops of three months duration requiring water for 30 days only.

(Mys. Univ. B. E. Civil Sept. 1956)

34. (a) Enumerate the staple crops under irrigation in the Bombay Deccan and give their approximate duty figures.

(b) Explain why the use of duty figures is not satisfactory in actual practice.

(Bom. Univ. B. E. Civil April 1956)

35. Write short notes bringing out its usefullness in irrigation practice on : Block System.

(Bom. Univ. B. E. Civil April 1956)

36. (a) Compare Lift Irrigation with Flow Irrigation and bring out clearly the advantages and the disadvantages of each of these methods.

(b) Discuss the several precautions necessary for increasing the duty of irrigation water.

(Mys. Univ. B. E. Civil April 1957)

37. Explain clearly and comment on :

(i) Although Bajri, Rice, Jowar, Gram, Tur and Maize are seasonal crops, their requirements of waterings differ.

(ii) Abundant application of water to sugarcane crop may produce higher yields, but not more of sugar proportionately.

(iii) The irrigator, if given a choice, prefers to have Khapla (Wheat) to Ordinary Wheat Crop, and Sugarcane to an Orange garden, nowadays.

(Poona Univ. B. E. Civil April 1957)

38. Design a canal to satisfy the following crop requirements and sketch the section with balancing depth of cutting.

12 months cane	12000 acres
Kharif crops	15000 acres
Rabi crops	16000 acres
Hot weather fodder crops	5000 acres.

The soil through which the canal flows contains 2′ of Black Cotton Soil at the top and Muram below. Show the distribution of the soil in the banks. The canal is not lined.

(Poona Univ. B. E. Civil April 1957)

39. Write explanatory notes on : Block System of Irrigation.

(Poona Univ. B. E. Civil April 1957)

Problems : For problems and their solutions, read pages 9 to 12 of "Solutions of problems in Irrigation Engineering" by Shahane and Iyengar.

CHAPTER VII

INUNDATION, BANDHARA AND WELL IRRIGATION

(A) INUNDATION IRRIGATION.

Inundation Canals: Inundation canals usually flow in summer, when rivers are high. No Head-works or weirs need be constructed across the rivers. As the rivers, where inundation canals are in use, generally change their courses, the head of the canal or the off-take is also to be changed to suit the conditions. As there is no regulator for the river, the flow of the canal varies with the flow in the river. In the inundation canal, the water contains good fertilizing silt.

Inundation canals are of ancient origin and practice. There were a good many of these constructed in olden days and some of these have been recently improved or converted into perennial ones.

The bed level of the inundation canal at the head is generally at L. W. L. of the river, and the average depth generally allowed in the canal is 5 ft. (The maximum allowed is even 10 to 20 ft.) The bed slope of the canal varies from 6 to 15 inches per mile. The alignment of the inundation canal is generally sinuous, being excavated by ordinary cultivators.

Inundation canals take off directly from the river, and are excavated parallel to the fall of the country or the course of the river. Just as in other canals, there are branch and distributary canals also excavated for inundation canals.

2. Silting of Canals: The velocity in the inundation canal being less than in the river, silting generally takes place and this is more in the head reach of the canal. The depth of the silt deposited varies from 1 to 5 ft. at the head of the canal, and gradually decreases (in depth) lower down, and extends to about a mile or two. This silting is more, if the river edge on the upstream side of the head is eroding, and less, if the canal is taken from a creek of the river. The deposit of silt at the head of the canal has many disadvantages, the chief of which is, it reduces the supply in the canal below the normal, and consequently the standing crops suffer.

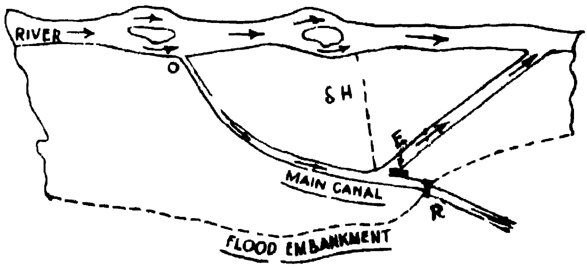
In such cases, a subsidiary head should generally be provided and opened when required, as the main function of an inundation canal is to supply enough water to the cultivators to sow their crops (indigo, sugarcane, and cotton) in the months of April and May.

To reduce the silt at the head, any sharp bends in the canal should be removed, the length of the canal shortened, and the gradient increased.

3. Selection of Site: (1) This should be as near the irrigated area as possible, the object being to reduce the length of the canal and the cost.

(2) The river course at this point should be straight. If in a curve, the head should be situated on the convex side, to avoid silt trouble.

GENERAL SITE PLAN OF INUNDATION CANAL



O—Point of off-take. S H—Subsidiary Head
E—Escape R—Regulator.

Fig. 29.

(3) The bed and bank should be stable and the bank should be high.

(4) The river should have a normal width and velocity, so that the variation of the depth of water should be as small as possible.

(5) The best site is on a by-pass or a creek, and the exact point is just above the junction of the by-pass with the main river, for the following reasons : (See Fig. 29).

(*a*) The bed of the creek (*Dhund*) is generally higher than the bed of the main river, so the heavy rolling silt is not found here.

(*b*) The velocity of the *Dhund* is lower, and so even small particles of silt drop down here.

(*c*) The dhund is generally on the other side of the curve of main river, and as such, it draws less silt.

(6) A site in a back-water is generally good.

(7) A site where erosion is occurring should be avoided.

4. Conditions of the River to locate an Inundation Canal. (1) To supply water to the crops, the first flood in the river should be an early one.

(2) The level of F. S. L. or as it is strictly called the "*fair irrigation level*" of the canal, should last for some good time.

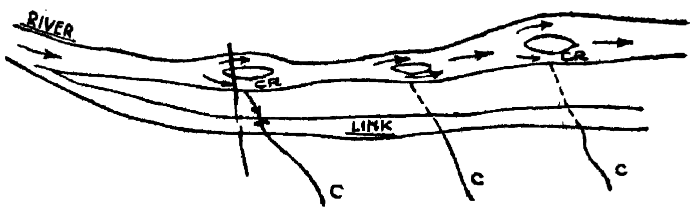
(3) The last rise of the river should be late, and must be useful to the crops (so that the Rabi crops may also be benefitted.)

Subsidiary Heads: The supply from the head should be uniform both for sowing and maturing the crop. The main head should be longer than the subsidiary head, because silt from the main head may not go below the junction of the subsidiary one. The subsidiary head should generally be boxed up, until it is required for use. When the subsidiary head is opened, the main head may be closed.

5. Bund For Canal Head (in Winter): This is to prevent rain water (from accidental rains) entering the canal. The bund should be put parallel to the river edge, and constructed on the sandy deposit and not on grass or silt. When opening the canal, a mere cut is made and water allowed into it.

6. Linking of Inundation Canals: As a satisfactory head is not easily got for an inundation canal, a remedy usually adopted is the linking of the canals. (See Fig. 30).

LINKING UP OF INUNDATION CANALS



CR—CREEK C—CANAL

Fig. 30.

This arrangement has the following advantages :—

(i) If one inundation has a good head, the others can also receive the benefit (by the link).

(ii) Due to the large carrying capacity of the bigger canal, there will be less silting.

(iii) The feeder of all the canals will have a more direct course and a flatter gradient than the river, and so it can supply the off-taking canals at a higher level than if these canals were taken direct.

The disadvantage in the linking system is that, if there is a breach in the feeder, all the canals will suffer.

7. Regulator: A few miles from the off-take, a regulator of masonry is sometimes constructed to pass off the excess water from the canal. The object of placing it at a distance is,

(i) The river course may change.

(ii) There may be scour near the river, and the regulator itself washed away during flood. Just above the regulator an escape is provided leading the water back to the river. Vide Fig. 29.

The banks of the canals are well maintained to serve as inspection paths, and planted with trees also for shade. These trees may provide fruit or fuel and give some income to the Government.

8. Escape: Near the flood regulator, on its upstream side, an escape is usually provided. It should be large enough to carry all the surplus water from the canal; and its alignment should be inclined downstream side at about 45° to the general direction of the river. Vide Fig. 29.

9. Design of the Inundation Canal: The full supply level of the canal is fixed at the level at which the water is more or less steady for a good number of days during the inundation season (40 to 50 days). The bed is also fixed as low as practicable. This is to make use of as much :of river water and for a longer period. These two things being known the section of the canal may be properly designed after fixing the bed slope.

The main point to be remembered here is: The cross-section should be kept liberal, because,

(i) While the Kharif crops are yet standing on the land, the Rabi crops are also to be watered.

(ii) The time factor is low in Inundation Irrigation (and hence a larger cross-section is required).

(iii) The water supply into the canal is not constant and cannot also be relied upon. For this purpose, the distribution of water is by rotation system so that water may be economically used.

In the design of the Inundation canal, the canal is made deeper and narrower. That is B/D is made small.

10. Works to be Attended to in an Inundation canal :

(1) Erosion is most common in the head reaches and this should be stopped by bushing, and sometimes by fascines and matresses.

(2) Silt clearing is to be done after every season is over and whenever circumstances permit.

(3) To convey the produce bridges for streams or nalas and roads are necessary, and will have to be constructed where required.

11. Crops Grown :

(1) Kharif crops: Rice, Bajra, Jowar and Cotton.

(2) Rabi crop: Wheat.

By the end of the inundation season, the land is flooded to grow wheat and the subsequent watering is got either from the open wells or from the canal, using Hurlo (a kind of Persian wheel).

12. Duty under the Inundation canal : Duty under an inundation canal is generally low, being 70 as against 200 in a perennial canal. This low duty is due to

(1) Sandy and porous soil.

(2) The short period of the crop—only 5 months.

(3) Low rain-fall during the season.

(4) Long water courses.

(The change of an inundation canal is generally begun from the flood regulator.)

N. B. The discharge in the canal varies from a few cusecs to seven thousand cusecs.

13. Advantages of Inundation Irrigation :

(1) As there are no head works in general use, the initial cost is much reduced.

(2) The water has good manurial properties, as it carries fine silt in suspension.

(3) As the supply in the canal is non-perennial, there is less trouble from water-logging.

14. Disadvantages of Inundation Irrigation :

(1) As there is no head work across the river, if the river changes its course, there is damage either to the head-reach or to the regulator.

(2) Supply of water is variable and unreliable. To overcome this, the cross-section of the canal is made bigger and even extravagant and this means extra cost.

(3) As the water supply is variable and uncertain, the cultivators use it in plenty (more than required) when available, and hence the duty realised is very low.

(4) As the supply is fluctuating, the cultivator does not much care to improve his land.

(5) As a silt-pond cannot be provided at the head, as in the case of a weir, the silt is passed on easily into the canal. A good amount of money is to be spent on silt clearing.

(6) As the time factor is small, the distributaries will have to be of larger size and hence more cost.

(7) Due to the rotation system being necessary to be introduced, malpractices are likely to be resorted to.

Owing to these many disadvantages, the inundation canals are recently remodelled and converted into perennial canals.

Delta Irrigation

1. Delta is the land formed, especially at the mouth of a river, due to overflowing its banks or *inundation* as it is called.

The Delta Rivers in Madras are the Godavari, the Krishna and the Cauvery. These rivers have their sources in the Western Ghats and flow towards the east. To reach the sea, they make their way through the Eastern Ghats and carry with them a lot of detritus. The Eastern Ghats are about fifty miles from the sea in the case of the Godavari and the Krishna, and hundred miles in the

case of the Cauvery. It is said that the sea was once washing the slopes of these Eastern Ghats and has gradually receded, as the river deposited its alluvial soil into the sea and began extending and reclaiming the portion; and it is said that this process is still going on.

2. Formation of a Delta : The river deposits the alluvial soil and forms a new ground, as the largest amount of deposit always takes place where its velocity is first lost, the ground in the immediate vicinity of the river gradually rises and forms its banks. During the succeeding freshes, the banks rise higher and higher at the margin, until at last, they are only occasionally overtopped (in floods) and so, the silt is carried further and further always in the same manner—making fresh land and spreading out in a fan-like shape. In course of time, as the direct course to the sea becomes longer, the overflow in the river naturally follows the direction of the greatest slope, which is more or less perpendicular to the river banks, and thus the river cuts new courses for itself and it is generally found that the branches of a delta-river multiply, as it approches the sea.

In the delta rivers, the slope of the river bed is greatest at the hill or highground, wherefrom it cuts away and descends into the plains, and gradually becomes flatter, when it reaches the sea. So, as a delta extends, the course of the river to the sea gets longer and longer and the slope of the bed and surface of the river gradually become flatter, until finally near the sea, it is very small. When the slope has become small, to discharge its volume, the river should increase its area (i. e., breadth and depth) and so it (rises) increases in depth and overflows its banks, and changes its regime and new branches are formed. Thus the original course becomes choked up with sand and other deposits, and it is impossible to distinguish between the main river and its branches and to regulate the water supply.

Peculiarities : In some of the branches of the rivers, there is a gradual diminution of the area of discharge of the river. (It is seen in the Mahanadi that in a distance of twenty-five miles, the area is reduced to half). The river overflows the banks in floods, but as the bank work here is done systematically and is maintained, there is less trouble.

In these delta rivers, the slope of the river is not uniform. It has sudden steeps and then flats. Again the surface of the river is also not uniform with the bed. But the bank is more uniform and it decreases in section from head to the sea. So it forces more volume to the lower reaches, and here the banks are low and the river overflows. Thus it is, that special attention should be paid to the protection of the embankments in the lower reaches.

3. Characteristics of a Delta River :

1. In high floods, it rises more or less above its natural banks.
2. Its bed slope gradually decreases as it approaches the sea.
3. The slope of the water surface also decreases (it does not follow the bed slope).
4. The sectional area of the river diminishes at a certain distance from the head of the delta.
5. Its bed runs on a higher level than the country.

4. Configuration of a Delta: A delta consists of a series of ridges and hollows of greater or less width apart, ordinarily lying in a direction more or less oblique to the river.

In some instances, there are ridges (their direction is more or less parallel to the coast line), which were originally beaches left by the receding sea. Advantage is taken of these ridges, and irrigation channels are taken along them and the low portion on both sides easily irrigated; also the hollow portions are used as drainage channels to carry surplus water to a lower level or even the sea.

5. Preparation of a Delta Project: The works ~~included~~ are:

(1) Protection from Inundation.
(2) Irrigation.
(3) Drainage.
(4) Navigation.

(1) *Protection from Inundation*: In investigating a delta project one is to study

(a) The extent to which the river and its branches are subject to inundation floods and

(b) The practicability of controlling them.

The inundation floods no doubt improve the lands near the river, which are high by their alluvium, but as they flow to the lower

lands, their volume and velocity increase and as they can only pass on slowly over the low lying lands, these latter become submerged for long periods and the crops are utterly destroyed. Hence the lands should be protected from inundation.

For this purpose, the following information regarding floods, should be collected:

(a) Periodicity

(b) Duration.

(c) Height.

(d) Number of floods annually, and at various points of the river, i. e., above the delta, at delta and below delta.

(e) The maximum discharge in ordinary floods, high floods and extraordinary floods.

Again, information regarding

(a) The longitudinal slope of the bed of the river,

(b) the surface fall of the river,

(c) the sectional area of the river, and

(d) the longitudinal slope of the country should also be collected.

From the above information, the capacities of all the branches of the river should be calculated and compared with the total discharge in the main river, and from the result obtained, the flood embankment should be designed.

It must be borne in mind, that when the upper course of a river is restricted by embankments, and no inundation allowed, the water will rise in the lower reaches, and unless proper precautions are taken, the country here will become deeply swamped and much damage will result.

Along with the construction of the embankment, the regime of the river must be strictly maintained and its course regulated by protecting its banks.

Construction of Embankments: In Northern India, the soil available for embankment is not good, so that the embankment gives way easily when there is any erosion; but in Southern India, stoney clay is available and the embankment is better fitted; even here it is sometimes protected by groynes of palmyra trees and matting (as in Tanjore) and stone pitching (as in the Godavari and Krishna Deltas). Sometimes, long grass or any tree which allows roots to bind the soil together, is allowed to grow or planted.

The dimensions of the embankment depend on the quality of the soil used. The top level of the embankment is fixed generally at about 3 ft. above H. F. L.

The slopes of the embankment are usually 2/1 on the water side and $1\frac{1}{2}/1$ on the rear side, which is generally protected with turfing. The width at the top also depends on the height of embankment, but generally a width of 8′ to 9′ is allowed, for cattle and men to use. An allowance of about 2″ to a foot is made for settlement in a new embankment.

If there is no channel parallel to the embankment for supply of water to the lands, sluices at intervals are constructed for the benefit of the ryots to get the alluvium carried in the floods.

(2) *Irrigation:* Most of the area in a delta is easily irrigated as the bed of the river is in a higher level than the adjacent lands, and the branches, when taken from it perpendicular to the main river, meet the ground level at a small distance.

When the depth of water in the river is low, and the banks at the Delta head are high, a canal to irrigate the lands will have to be taken in deep cutting and also have a great width (because depth is small). Hence if an anikat is built here, and the water level in the river thus raised, the depth in the channel will be more (sectional area may be made less) and this will facilitate also Navigation.

Alignment of Canals and Branches: Two main canals are taken, one from either side, mostly in straight bits to reach the ground level, as near as possible. From this point, a branch channel is taken parallel to the river bank and constructed up to the river mouth, or the tidal water, and this branch channel is generally made navigable. One or more branches are also taken from here, mostly on ridges to command as much area as possible, and the main-line itself is also carried on, skirting the edge of the Delta, until it meets the tidal water.

(3) *Drainage:* Drainage is very important. If this is not done, more harm is done to the crop than good.

Soil becomes sour and roots become destroyed.

It does not matter if more water is allowed, but it must run away.

(4) *Navigation*: Cheap carriage is essential, especially in a poor country like India and we know that carriage on water is cheaper than on land. Of course, there is extra cost, if the canal is also to be used for navigation; but the corresponding advantages are so much, that this additional cost can be ignored.

Plantations: Irrigation works afford in many cases the best possible opportunity of growing trees for firewood (fuel) and other purposes, at a nominal cost.

Road avenues cost a good deal to form and establish, and generally need nurseries and special superintendence. On irrigation work, it is different--The soil is generally suitable; the necesssary moisture is present here and the trees grow easily and rapidly. Handsome shady trees which are essential to road avenues are not necessary on irrigation works.

Generally, hard trees such as Bomboos, Babul, etc., are easily grown. None of the varieties of Bannyan should be grown on irrigation works, (and generally wide root-spreading trees should not be allowed near any masonry work.)

6. Fencing. The common aloe fencing is the best and the cheapest. Cattle should not be allowed to graze on the canal bund, the grass should only be cut and sold.

(B) BANDHERA IRRIGATION

1. General: This system is practised on a large scale in some districts of the Bombay State (Parts of Poona, Khandesh and Nasik).

Bandharas consist of a series of masonry weirs constructed across a river at distances of a few miles from each other. A small canal with a regulator at its head, is taken off from each of the weirs. These channels or canals vary in length from one to five miles, and terminate in what is called *Thal* or low lying land, which is watered by the Bandhara for the benefit of the cultivators.

This is the cheapest type of irrigation and has been in use for a long time. This type is specially useful where small isolated catchments cannot be economically made use of otherwise.

The area irrigated under a Bandhara varies from a few acres to even a thousand acres.

A Bandhara can be successfully constructed across a stream (though small) which has flow in it, throughout the monsoon season and up to Jannary. If the stream is perennial, it can be used for crops like sugarcane and other perennial crops also.

Bandharas are mostly used for monsoon crops, though sometimes advantage is taken of it, for summer crops.

(Some of the Bandharas have been provided with reservoirs so that they may be useful throughout the year.)

2. Selection of the site of Bandhara : The selection of the site for a Bandhara is similar to the selection of site for any weir constructed across a stream or river, to lift the water level and use the water for irrigation.

The conditions are

(1) The river or stream should be perennial, so that the lands may get the required water even during summer.

(2) There should be good rocky foundation available at a high level for the masonry wall.

(3) The site should be such, that the river has a drop in its bed at the place, so that there may be free over-fall over the crest avoiding any afflux or heading-up.

(4) The H. F. L. at the site and the level of the crest should be as low as possible.

(5) The section of the stream should be narrow at the site.

(6) Above the site of the Bandhara, the stream should have a gentle slope with its tributaries joining it, and it should be fan-shaped to have a good storage.

(7) The stream should have a good fall down-stream of Bandhara and the area commanded also fairly steep, so that the canal may get a greater command.

(8) Heavy and costly cross drainage works should be avoided.

(9) The site should be selected with regard to the level at which the water has to be delivered to the land.

(10) The commanded area should be fertile, well drained, compact and near to the Bandhara, to reduce the loss of water during transportation and to reduce the cost (due to the shorter length of the canal.)

(11) Sufficient labour should be available near by, to cultivate the lands.

(12) The natural banks of the stream should be high, so that during floods, there may not be much submersion of the marginal lands.

3. Determination of the Section of Bandhara: The Bandhara is like an ordinary pick-up weir and hence the section of the Bandhara-wall should be designed like any masonry weir. As usual with the design of any weir wall, the following information should be collected :

(1) Trial bores are excavated at the site, and the levels of the hard rock at the proposed site found out. To take advantage of the rock, the alignment is generally made to suit formation of the hard rock level, so that it need not be in one continuous straight line.

(2) The high flood level of the river and the full supply level of the canal was taken into consideration, and the crest level of the Bandhara and the bed level of the canal are fixed.

(3) Knowing the crest level of the Bandhara wall and the deepest point of the foundation of the rock, the section of the Bandhara is calculated by means of stability diagrams.

For this purpose, the pressure at the toe of the wall (both upstream and downstream) should be taken for both the following conditions :

(a) When the upstream water is at the crest level and the downstream is dry.

(b) When the weir or Bandhara is submerged at high flood level.

SECTION OF BANDHARA WALL

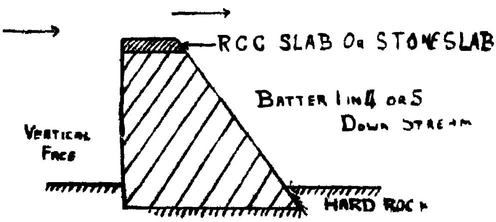


Fig. 31.

For working out stability diagrams the specific gravity of masonry is taken as $2\frac{1}{4}$ or $2\frac{1}{3}$ according to the kind of masonry used.

As a practical measure. the minimum top width is kept at 4 ft. The front face is made vertical, and the rear face is given a slope varying from 1 : 2 to 1 : 5. (Vide Fig. 31). Where the Bandhara is made use of by ryots as a passage or footpath (when the river is not in floods) the top width is made wider to suit their requirements.

N. B.— If the wall is to be raised on a subsequent date, either temporarily to store water for the dry season, or permanently, provision should be made for sufficient top width of the wall.

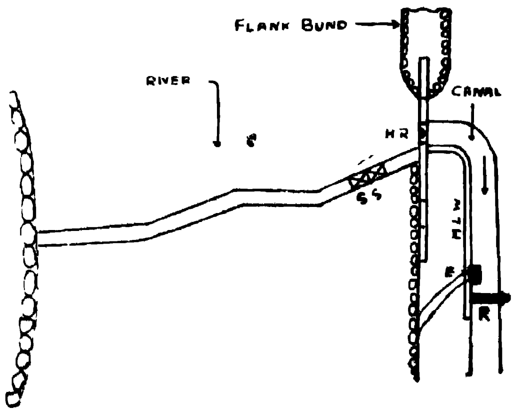
The Bandhara is treated as a broad-crested weir and the usual formula $Q = 3 \cdot 1\ LH^{3/2}$ used to find the length of the weir required.

4. Construction of Bandhara : The weir or Bandhara is never straight but winding. This is to take advantage of the rock outcrop in the bed.

To get rid of sand and silt accumulated in front, a sluice with an opening of about 3 ft. × 4 ft. is left at intervals in the wall.

On the upstream of the Bandhara, to prevent flood water outflanking, necessary masonry walls and training banks or flank

BANDHARA, GENERAL SKETCH PLAN



H. R.—Head Regulator, S. S.—Scouring Sluice.
M. L. W.—Masonry Lining Wall. E.—Escape. R.–Regulator.
Fig. 32.

bunds are constructed, the top of the wall and the embankment being kept 3 ft. above the H. F. L.

For the regulator which leads water from the river into the canal, a wall is constructed on the down-stream side, especially to prevent the down-stream flood entering into the canal, by having the top of the wall well above the H. F. L. Sometimes a scouring sluice or escape is also provided at the tail end of this masonry wall, to remove the silt generally deposited at the head. Beyond this scouring-sluice, a regulator is also provided. Vide Fig. 32.

5. Canals from Bandhara : The canal from a Bandhara is similar to the one from any ordinary weir, only it is *kacha* and the alignment is very rough and is taken to suit the conveniences of the ryots. The outlets in the canal are also temporary. In some cases, where the main canal is big, distributaries are taken from it.

6. System of Irrigation under Bandhara : As adopted in the Bombay State, the irrigated area under the canal from a Bandhara is called *Thal*. It is divided into small parts called *Phads*, and each phad contains a number of agricultural plots belonging to different ryots.

The only condition of irrigation is, that only one kind of crop should be grown in one complete phad during the year. In other phads, other kinds of crops may be grown as per rotation. This system is called *Phad System*.

7. Maintenance of the Works Pertaining to Bandhara : All ordinary repair works will have to be carried out by the royts or irrigators only. Any special or major works will be done by the P. W. D. and a contribution (about one-third) is demanded from the ryots.

8. Advantages of Bandhara Irrigation : (1) As the area is compact, and the irrigation is intense, a high duty is realised.

(2) Small catchments which were otherwise going to waste are utilised for the benefit of the ryots.

(3) It is of immense help, especially during the present food shortage in India.

9. Disadvantages of Bandhara Irrigation : (1) The irrigated area is fixed and even when more water is available, it cannot be made use of.

(2) As small catchments having even non-perennial streams are attacked, supply of water to the lands is not satisfactory.

C. Well Irrigation.

1. Sources of Water Supply : Atmospheric precipitation is the ultimate source of all water supply. This may be divided into

(1) *Underground Supply* : This is the portion which percolates into the ground. This underground supply gives rise to (a) Natural Springs, (b) Karezes, and (c) Wells.

(2) *Overground Supply* : This portion flows on the surface as run-off. Part of this overground water goes into the rivers, and part is held as surface collections in lakes, pools, tanks, etc. (This is not treated in this chapter).

(1) *Underground Supply.*

(a) Natural Springs : (2) When water collects under pressure between two impervious layers sandwitching between them pervious layers of soil, springs are formed. These can be seen in valleys where the water, which percolates through pervious strata in the high ground forming a reservoir below the valley, comes out of the fissures etc., in the upper stratum of the valley.

3. Spring Wells : Open wells in rocky strata are called spring wells. These can be found therefore, in hilly undulating tracts and in places with rocky substratum. They get their water-supply from the water that gets in through fissures, joints and faults in the rocky strata. The ideal position for a spring well is at the junction of major joints, and hence for the successful sinking of a well in rocky strata, such joints are to be located first. The quantity of supply from the well will be more in highly fissured rocks. (The fissure should be only in the upper layer and the lower layer should be water-tight). In this respect, slates are better than shales due to joints, and marbles are better than limestone rocks. Igneous rocks like gneisses and schists are generally disappointing.

As the rate of infilteration of water in these rocks is very slow. depleted wells do not fill quickly, and take a long time. To increase the supply of such wells, infiltration galleries are often driven.

4. Artesian Wells : These occur in places where artesian conditions exist, the essential conditions being,

(i) The existence of a porous bed into which water can freely percolate.

(ii) The complete imprisonment of this water-bearing porous stratum, between two impervious strata.

(iii) A difference in height between the place at which the water enters the porous bed and the point below at which this porous bed is to be tapped by boring.

Artesian wells can be dug only when the above peculiar geological formation exists. Artesian wells are to be found in South India near Pondichery, and Kathiawar and Gujerat, and Quetta; however, it may be said, that no truly artesian conditions have so far been met with in India.

The thickness of the porous rock, its extent and porosity, determine the quantity of water. The difference in height between the depth at which the water is tapped and the outcrop, and the freedom with which the underground water can percolate, determine the pressure.

Artesian wells may be classified into,

(i) Fully Artesian or flowing wells, and (ii) Semi or sub-artesian wells. (See Fig. 33).

FULLY ARTESIAN AND SEMI-ARTESIAN WELLS

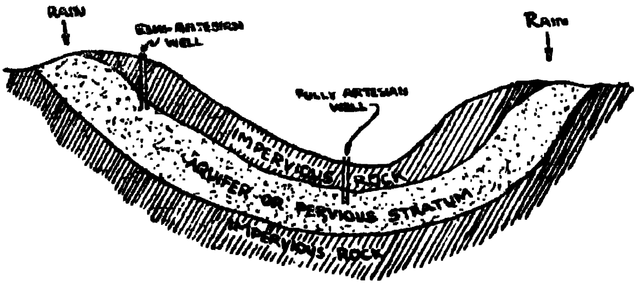


Fig. 33.

Fully Artesian Wells: Here the underground water does not require lifting or pumping up, because the water is under great pressure and rises to the surface of the ground (and in many cases up to a great height), and keeps continuously flowing. By "Artesian Wells" these flowing wells are usually meant.

Semi-artesian Wells: In this case, the water does not come up the bore of its own accord, and hence water lifting devices have to be used.

When artesian wells are used to irrigate valuable crops, the supply of water from the well is first stored near by and water is taken from this storage, whenever required.

At the present, so far as irrigation in India is concerned, artesian wells are of very little importance.

(b) **Karezes:** (5) In Baluchistan where the rainfall is scanty and very capricious, irrigation has been effected from time immemorial by a system known as Kareze. The country around Quetta and the town itself, is in a line with small streams of water which are derived from tunnels driven into the hillside which trap underground springs; each of these tunnels is called *Karez* and is generally only just large enough to permit a man to creep through it.

The Karezes are connected by sinking a shaft from the surface about 50–80 ft. apart, on the line which the Karez is designed to follow, and thus connecting the bottom of the shaft by a tunnel (about 3 ft. × 20″). The tunnel has a bed fall, so that the water may flow by gravity into a channel at its outer toe just where it emerges from the rock.

Tunnels are sometimes driven in sloping side of hill country to tap the subterranean water supplies. These are practically horizontal wells, differing from ordinary wells in that water has not to be pumped to bring it to the level of the surface, but finds its way by gravity-flow to the lands on which it is to be utilised (See Fig. 34).

(LONGITUDINAL SECTION) K = KAREZ

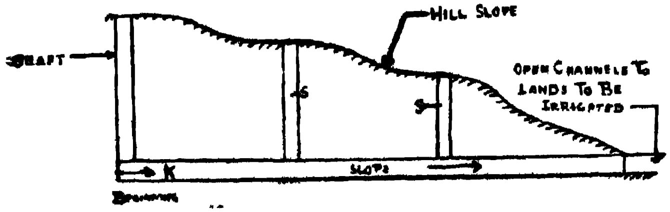


Fig. 34.

Examples: (i) Near the Khojak Pass is a great tunnel of this type—The Zaria Karez. It is run near the dry bed of a stream into the gravels, for a distance of over a mile. It is 70 ft. deep at the first shaft. The slope of its bed is 1/1000, and its cross section is 1·7 × 3 ft. and the discharge 9 cusecs.

ii) The water supply is given to a colony from a tunnel 3300 ft. in length, the cross-section having height $5\frac{1}{2}$ ft. width $3\frac{1}{2}$ at bottom and 2 ft. at top and discharges being 6 cusecs.

(c) WELLS

6. Wells: A part of the rainfall sinks into the soil, the amount depending upon the nature of the soil. When the rainwater sinking under-ground meets an impervious stratum, it is detained there, and forms the under-ground water table. This sub-soil water has lateral movement from one part of the sub-soil to the other. It must be understood that these under-ground water reservoirs are not hollows filled with water, but consist of soil saturated with water. To use this sub-soil water for irrigation purposes, water lifting devices are usually employed. Thus sub-soil water irrigation is mostly a form of lift irrigation.

Sub-soil water for irrigation purposes is usually available in the form of wells and Karezes.

Wells are holes sunk from the ground level into the sub-soil water-table to utilise the under-ground water.

Irrigation by wells is known as *well irrigation*. In India, well irrigation forms more than 1/3 of the total irrigated area, and it is estimated that the value of the out-turn from well irrigation is from 1/3 to one half of that of the whole irrigated crops. Thus *well irrigation is an important system of irrigation in India.*

Wells may be classified into,

(i) Open wells, (ii) Bore-wells.

OPEN-WELLS

7. Open Wells: Open wells are more commonly used. These have a relatively bigger diameter. When open wells are dug in alluvial soil, they are often lined with bricks from inside. For an ordinary open well with impervious lining in an alluvial soil, the maximum safe discharge is taken to be 0·15 cusecs. The well

is usually constructed with its bottom about 20 ft. below the water-table.

The supply of water is more *reliable* in alluvial soil though the construction of a well in such a soil is difficult and of comparatively high cost. If the soil is hard or rocky, *steining* may not be necessary. In soft and porous non-alluvial soil, lining is necessary. Even in hard non-alluvial soil, lining or steining is advised and the steining can be generally done after the well has been completely dug. Generally speaking a well with lining, though costlier, is to be preferred to a well without lining.

Open wells can be constructed economically up to a depth of 100 ft. In alluvial soils, wells without lining are practicable only up to depths of about 20 ft. maximum.

Wells are sometimes classified as follows (desending upon the nature of the lining).

(1) *Kacha wells*: These are wells without lining, and are generally of a temporary nature. They are possible only when the spring level is not below 10 to 15 ft. from the natural surface, and the soil can stand vertically without lining. A *Rati or Picottah* is generally used to lift water from these Kacha wells.

(2) *Pervious lined wells*: Pervious lining is also used for some wells. These are also of a temporary nature. Here the water gets into the wells from the sides also. Water from these wells is lifted by means of a *Denkli.*

(3) *Pucca Wells*: These are imperviously lined wells and are the common types of wells met with. This class is the most suitable for irrigation, as it is not liable to deterioration. For 50 ft. deep wells, the lining is usually $1\frac{1}{2}$ ft. thick. Below 50 ft. it is made 2 ft. thick. (Lining is done simultaneously with the construction of the well). Wells with impervious lining can be dug to a comparatively greater depth than wells with pervious lining. In the case of imperviously lined wells, the water can percolate into the well from the bottom only.

8. Yield of an Open Well: Regarding water from wells used for irrigation, two things are essential.

(1) Quantity and (2) Quality.

(1) The **quantity** of water available in any well is **tested** as noted below, and if the quantity is sufficient, the well is fit for

irrigation. One precaution to be taken here is, the quantity available should be tested at the driest period of the year, so that the required quantity of water may be available for irrigation at all times.

To test the yield or capacity of well, two methods are generally employed.

(*a*) The Pumping Test, (*b*) The Recuperation Test.

(*a*) *Pumping Test*: In this test, the water level in the well is depressed or lowered below normal, to a safe maximum working head, and the yield at this level measured by the quantity that is baled out by pumping, in a given time. This level must be maintained even after pumping, so that the infiltration of the water into the well is equal to the quantity pumped out.

This test is difficult, because (1) steady pumping to maintain the water level is difficult; also (2) accurate measurement of the discharge from the pumps is not practicable.

The maximum yield from well is obtained when it is worked out continuously without exceeding the critical velocity.

N. B. Instead of taking the discharge directly from the pump or, if ordinary baling is done, instead of calculating the discharge by noting the time and the quantity removed, it is better if the discharge is gauged on a weir.

(b) Recuperation Test: The water in the well is pumped to any desired level below normal, and pumping is stopped for a while, and the level of the water in the well allowed to rise or to refill to the original level. The time taken and the quantity filled in, are both accurately measured and the discharge arrived at.

This method is more advantageous than the pumping test method, because,

(i) This gives more reliable results than the other.

(ii) If the sub-soil is sand alluvium, the yield is uniform all over the surface.

(iii) It is easy to note the timings and the level of water.

The defect in this method is, the rate of filling is more rapid when the head is greater, and it becomes less as the water level rises up.

9. (2) Quality or Salinity in well water: Regarding the quality of water required for irrigation, there are some salts

dissolved in water, which affect the growth of plant and thus the irrigation. In other words, the presence of salts or the *Salinity* of the water must be tested before use, and any water should not be indiscriminately used for irrigation.

The salts commonly met with in water are,

(a) Sodium Salts : These cause base reaction and make the water alkaline.

(b) Calcium Salts : These salts do not prevent any reaction but only delay it.

The Irrigation Research Institute at Lahore has given the salt index for any irrigation water as follows.

Salt Index = (Total Na − 24·5) − (Total Ca − Ca in $CaCo_3$) 4·85. The quantities referred to here are parts per one lakh units.

If the result is negative, the water is suitable for irrigation, and if it is positive, it is unsuitable.

N. B. Brackish water is generally unfit for irrigation, and sweet water good for irrigation.

10. Determination of Specific Yield or Capacity of a Well : The discharge from a well varies directly with the depression head; that is, Q varies as H, or Q = KH, where K is a constant.

In Fig. No. 35 let *aa* represent the original water level and

WELL FINDING DISCHARGE

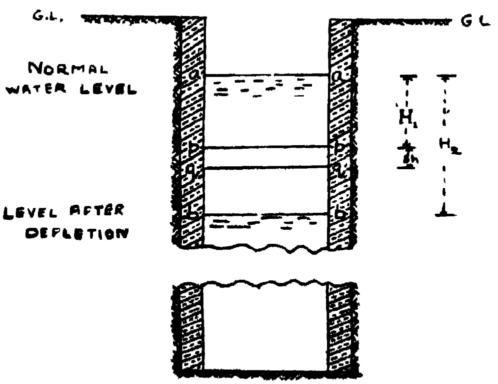


Fig. 35.

bb the level of the water in the well after pumping. Let *pp* be the water level at any time, say t, after the stoppage of pumping. If *qq* is the level at a time $t + dt$, then the quantity recuperated into the well in time dt is given by $q = A \times dh$, where A = area of the well, and h = height.

Again, $q = Q \cdot dt$

Therefore $A \cdot dh = Q \cdot dt$

But $Q = K \cdot h$ as per equation (1) above

Therefore $A \cdot dh = K \cdot h \cdot dt$

or $$\frac{dh}{h} = \frac{K\,dt}{A}$$

Now integrating between $t = 0$ and $t = T$, when $h = H_1$ and H_2, we get

$$\log_e \frac{H_1}{H_2} = \frac{K}{A} T, \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

or, converting into common logarithms,

$$\frac{K}{A} = \frac{2{\cdot}303}{T} \log \frac{H_1}{H_2} \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

where, H_1 and H_2 are the depths of water below normal water level in time T.

$$K = 2{\cdot}303 \frac{A}{T} \log \frac{H_1}{H_2} \quad \ldots \quad \ldots \quad \ldots \quad (3)$$

K is called the specific yield or the specific capacity of the well.

The specific yield of a well is defined as the yield of the well in c. ft. per hour under a head of one ft.

K/A is the yield in c. ft. per hour per sq. ft. of the area of the well with a constant head of 1 ft. K/A is got from several experiments (as per actual tests), and the figures for different soils are noted below.

Soil		K/A
Clay	...	0·25
Fine Sand	...	0·50
Coarse Sand	...	1·00

Note—If K/A is constant in a well when several tests are made, it may be presumed that the critical velocity is not exceeded.

The actual velocity as worked out, should always be less than the critical velocity.

$$Q = Kh \quad \text{but} \quad Q = AV$$

$$\text{or} \qquad AV = Kh.$$

$$\text{Therefore,} \quad V = Kh/A.$$

Here we know K from equation (3) and h and A are known: hence V should be calculated, and seen if it is not less than the critical velocity.

11. Modes of Lifting Water:—The water required for irrigation is lifted either by (a) Man or animal power, or (b) Mechanical power and (c) Power from nature.

(a) *Man or Animal Power*: This was the original or primeval method adopted for lifting water and is still in use in many parts of India. Now-a-days they are being improved or replaced by other methods.

The methods employing man or animal power for lifting water are

(1) Rati, Grini or Pulley.
(2) Denkli, Picottah or Lat or (Shadouf in Egypt).
(3) Basket (Scoop) or Natali.
(4) Doon.
(5) Archimedean Screw.
(6) Mote, Charus, Pur or Bag.
(7) Persian wheel or Sakiyehe.
(8) Chain Pump.

(1) **Rati** *(Pulley)*: Vide Fig. 36. This consists of a rope passing over a pulley which is fixed in a frame work over the well. To each end of the rope, a pot is attached, and it is filled up and emptied alternately by the man pulling the rope.

It is used only for small plots of lands. About 0·1 acre can be irrigated per year by one Rati.

RATI

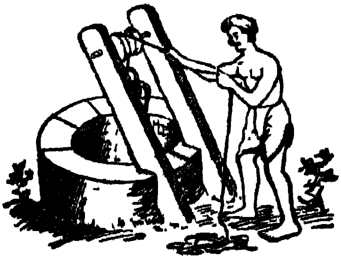


Fig. 36.

(2) **Denkli.** Vide Fig. 37. *(Picottah, Lat or lever)*: This is worked on the principle of the lever. It is suspended on a fulcrum and has got a counterpoised weight at one end (usually a clod of earth or a man). A bucket is suspended from the long end of the pole, and is made of leather. When the operator throws the weight on the sweep, the bucket fills, and the counter weight raises it and the water is poured out.

DENKLI

Fig. 37.

It is operated by one man who can easily lift it from 5 to 6 ft., but if it is arranged in series, the lift can be made even 20 to 24 ft.

One man can raise 22 gallons per hour from 5 to 6 ft. depth It is generally used for lifts of 4 to 10 ft. The efficiency of this method is 50 per cent, Small garden or vegetable plot (about one acre) can be irrigated by the Picottah. The initial cost here is small.

(3) *Swinging Basket or Natah*: This system is a very crude one and is resorted to only when the water is very shallow and not much below the level of the banks. A leather bucket or basket is swung on ropes held by two men standing opposite each other. It is dipped into the water and lifted and the water emptied into the water course.

BASKET

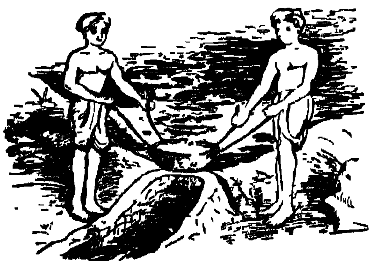


Fig. 38.

Two men can raise approximately 12 gallons per minute to a height of 3 to 4 ft. Assuming one man can do 1/8 H. P. work, the efficiency of the basket is only about 50 per cent.

(4) **Doon:** Doon is a wooden trough which oscillates on a fixed centre and one end of it is alternately dipped into water and raised. There is a counter-weight to balance the weight of water in the trough when filled. The man who operates it, stands on a plank in the water and depresses the end of the Doon and when the trough is filled, the man with his feet stepping on the plank, lifts the Doon slightly with his hand and allows the water to flow into the field channel.

DOON

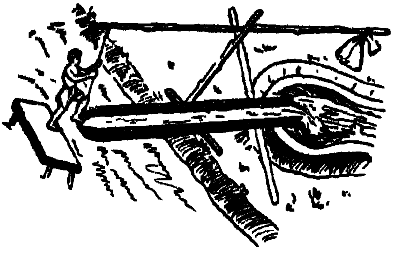


Fig. 39.

The Doon is generally used in Bengal and for a lift of 1 to 3 ft.

(5) **Archimedean Screw:** This is generally operated by hand, but in some parts of Egypt, small engines are used to drive it. The construction and operation are described below.

ARCHIMEDEAN SCREW

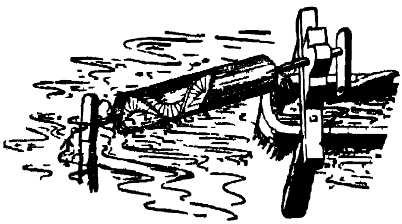


Fig. 40.

It consists of a tube revolving on an axis or a screw-vane on a shaft fitting into a cylinder. Around the shaft (about 14 or 15 ft long) is built a screw, made of thin pieces of wood fitted together as to be practically water-tight. A water-tight wooden cylinder is constructed around the screw. The diameter of the cylinder is ordinarily about 14 inches and its length does not often exceed 8 or 9 ft. The pitch of the screw is, about one revolution to 1¼ diameter. The screw is so attached, that it will not revolve on the shaft. The shaft projects from both the ends of the cylinder and is supported near its extremities by posts. The screw inclines at 30 degrees or less to the horizon, with its lower end in the water. To the upper end of the shaft a crank is attached.

Its action is simple. The screw in tube, when turned, transfers water from one end to the other. Each successive portion of

the tube occupies at one time, the position lower than that of the one immediately preceeding it, and must therefore receive or carry on the water that enters it at its lower extremity. In addition to this, the velocity of revolution will tend to force the water up. Or, in other words, when the handle at the upper end of the screw is turned, water moves up the hollow cylinder and discharges from the upper end of the cylinder into the field channel.

One or two men usually operate a screw. But in rare cases, when the screw is especially large and the lift considerable, a small engine is employed. High lifts are practically impossible on account of the difficulty of supporting a screw of great length.

One man can irrigate from one to two acres a day with the machine, provided the lift is not over two feet. The screw gives a larger duty for such lifts (3 to 4 times) than other arrangements.

The screw is used mostly in Egypt and some European Countries and the Middle East.

(6) Mote, Charus or Pur: Here the water is raised from the well by means of a leather bag with the help of two bullocks (sometimes two men are employed instead of bullocks).

At the site of the well a pulley is fixed to a frame overhanging the well, on which a rope moves, carrying at one end of it the mote. The other end is fixed to the yoke of the bullocks. An incline is made from the site of the frame, so that bullocks begin to move down, when the leather bag is filled with water; and when the bullocks reach the bottom, the leather bag comes to the top of the well. Now the driver cleverly manipulates the lower part of the mote and lets the water escape from the lower end of the bag into the trough and on to the field channel. Vide Fig. 41.

MOTE

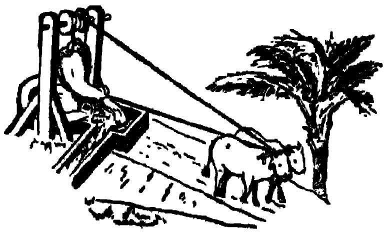


Fig. 41.

There are two systems of using the mote.

(a) Kili system. (b) Lagor system.

(a) **Kili System:** There are two ramps or inclines in this system. When the mote is filled with water, the bullocks go down the ramp and when they reach the bottom-most point the mote has come to the surface and the water is emptied. The driver removes the peg or Kili (that is why the system is called the *Kili* system) which fastens the rope to the yoke and holding the end of the rope in hand, allows the weight of the *pur* to draw him up the ramp. Meanwhile the bullocks walk up slowly along the parallel ramp and are ready at the top. Sometimes another pair of bullocks will be waiting at the top to give rest to the original pair.

(b) Lagor System: Here there is only one ramp and the bullocks will have to walk back up the slope, with the rope still attached to the yoke.

Advantages of Kili system over Lagor: (1) It does not harass the bullocks. The *lagor* gives jerks to the neck of the bullocks, when the empty *pur* is thrown back into the well. There is no time for food for the bullocks; but in the *kili* system as a number of bullocks can be used, it is advantageous. (2) It is easier for the driver, as he is pulled up the ramp. (3) It enables a number of bullocks to be used at the same time, thus saving delay and expense.

This system is extensively used in Gujerat. The area per mote is two acres in black cotton soil, and four acres in the other soils. The average area that can be irrigated by a simple mote is three acres.

The mote can be used for lifts up to 30 ft. and has a discharge of 0·8 cusecs.

The area irrigated for different crops are

Crop	Area Irrigated
Sugarcane	2½ acres.
Jowar	3½ to 4 acres.
Rice	7 acres (supplemented by rain).

(7) Persian Wheel: This consists of a horizontal wooden wheel about 10 inches in diameter with legs projecting.

A horizontal wheel is geared to a large vertical wheel on a horizontal shaft, to the other end of which is also fastened another

large vertical wheel which carries earthern or iron pots or buckets (at intervals of 1 to 1½ ft.) to raise the water. The horizontal wheel is made to rotate by bullocks. The pots are attached to an endless rope which revolves on the wheel, or attached to the circumference of the wheel itself. Each pot delivers its contents into a trough (from which the field channel starts) and then descends to the well to be filled up again. Vide Fig. 42.

PERSIAN WHEEL

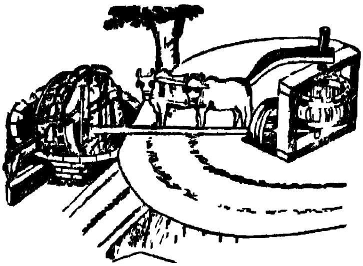


Fig. 42.

One man and two bullocks working for one hour can lift 125 c. ft. of water from a well 40 ft. deep.

Persian wheels are used in Northern India (especially Sind and the Punjab), Egypt, etc. They give better results when water is plentiful and near the surface.

The capacity of a Persian wheel ranges from 15 to 25 gallons per minute for lifts of from 8 to 12 ft. A Persian wheel can irrigate 25 to 30 acres and the lift varies from 20 to 50 ft. The discharge is 0·15 cusecs generally, as used in lined wells.

N. B.—The Persian wheel and the *mote* are the most effective methods for lifting water by manual power, as water can be raised from great depths. The Persian wheel is superior to the mote, as it supplies a continuous stream, but if the average is taken (for a lift of say 40 ft.) the total supply will be the same.

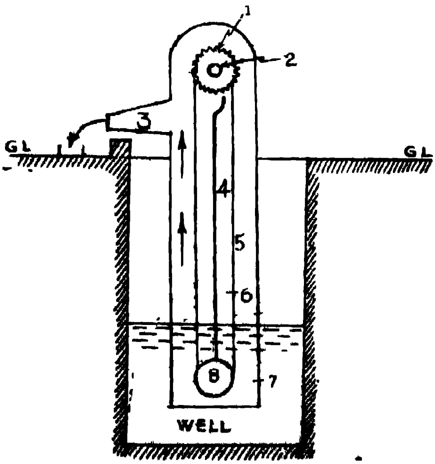
In Sindh, when the flow in a canal is low and not sufficient for the crops, a well is sometimes excavated in the middle of the berm of the canal, water is led into it and a persian wheel installed to lift and use the water for the lands under irrigation. The type of the Persian wheel used here is called '*Hurlo*'.

(8) Chain Pump : This is a device used for lifting water from shallow wells. It consists of an endless link belt working

over two rotating iron drums. The bottom drum is slightly below the water-level of the well and the other one is at the top of the well. A cylindrical iron casing partitioned into two compartments surrounds these drums. One compartment is a narrow cylindrical one and touches the inside of the casing. The lifted water comes out of a spout fixed at the top of this narrow cylindrical compartment. At intervals, along the endless chain, there are a number of floats attached. These are so arranged that they can just move up through the narrow cylindrical compartments, with a little clearance between them and the inside of this compartment. When the upper drum is rotated, water is caught up in the floats of the chain belts, and is discharged into a trough inside the revolving drum. While moving up, the floats are in a horizontal position and drag water over them; but while moving down the bigger compartment, they turn about a hinge near the chain and lie in a vertical position.

A handle is fixed to the upper drum, and the chain pump is operated by hand. A chain pump of ordinary size requires four men to work it, in two shifts. Its efficiency is low, and it is not generally used, because of the difficulties in getting it repaired, when it goes out of order. (Vide Fig. 43).

CHAIN PUMP



1. Upper Drum with clutch. 2. Handle. 3. Spout. 4. Partition. 5. Chain (Endless). 6. Float. 7. Outer casing. 8. Lower [illegible]

Fig. 43.

12. A comparative statement showing the efficiency of the different methods of lifting water is given below.

Lifting Device	Number of cft of water raised 1 ft high in 9 hours	Number of acres irrigated per year	Discharge in cft. per day of nine hours	Discharge in gallons per minute	Foot-tons per head per hour	Lift in ft.	Labour used	Duty
Rati	...	0·[illegible]0	615	35 71	32	4–17	1 Man	0 8
Picottah	3,300	0·91	416	12·00	22	7–15	1 Man	0·91
Basket	20178	...	...	35 to 70	20	...	2 Men	...
Mote	79200	3·47	2069	0 2 cusecs	59	20 to 60	Cattle 2	3·91
Doon	...	...	...	25 cusecs	20	...	1 Man	...
Persian wheel	...	...	1815	0 15 cusecs	57·1 to 100	10 to 60	Cattle 2	2·76
Archimedian Screw	...	...	...	25	20	...	1 Man	...
Chain Pump	...	...	...	...	30	15	1 Man	...

(b) **13. Modes of lifting water by Mechanical Power :–** The mechanical power used may be any one of the following. (i) Steam Engine. (ii) Oil Engine. (iii) Petrol Engine or (iv) Electrical Power.

To lift the water, a pump will have to be used and it may be (i) Airlift pump (ii) Force pump (iii) Pulsometer pump or (iv) Centrifugal pump. Of these, the air lift pump and the centrifugal pump are the most commonly used, and they are described below.

(1) **Air Lift Pumps :** Air lift pumps, though invented as long back as 1779, were not much in use, as they were not at first successful; but they have been revived and improved. The air lift pump consists of two parallel pipes, one for delivery and the other (of smaller diameter with cross-sectional area of about 1/7 to 1/5 of the delivery tube) for pumping compressed air. The delivery tube is also called the *education pipe*, and a good length of this pipe should be submerged below the water level in the well. The lower end of the air pipe terminates in a nozzle which enters

into the lower end of the vertical pipe. (See Fig. 44). The compressed air from the air compressing machine is forced through the

·AIR LIFT PUMP

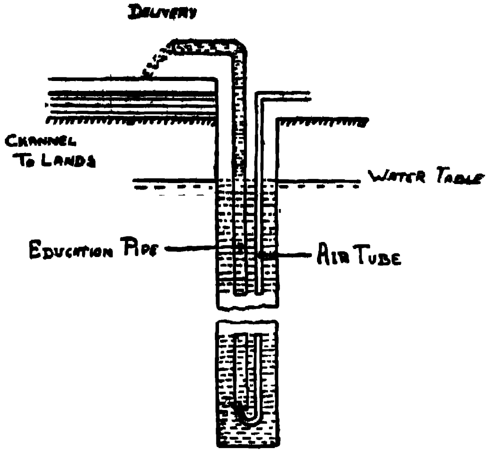


Fig. 44.

air pipe, to issue out from the submerged end in the bottom of the water pipe or the delivery pipe. The compressed air, after rising through the water main, gets incorporated in the water and causes the specific gravity of water to go down more and more, so that, due to the pressure at the bottom on account of the atmosphere and the water column, the level of the water in the delivery tube rises more and more (above the water level outside the water pipe) till the water inside the water pipe, rushes out of the discharging spout at the top. It is evident that the difference of pressure causing the flow, increases with the depth of submersion, so that for pumping with higher heads, more submersion is necessary.

There are three systems or methods of fixing the water pipe and the air pipe according to the requirements. (Vide Fig. 45)

(1) *The Phol System* : The water pipe and the air pipe are fixed side by side.

(2) *The Central System* : The air pipe is fixed inside the water pipe.

(3) *The Reservoir system* : The water pipe is surrounded by the air pipe and air is allowed through the space between the two.

AIR LIFTS

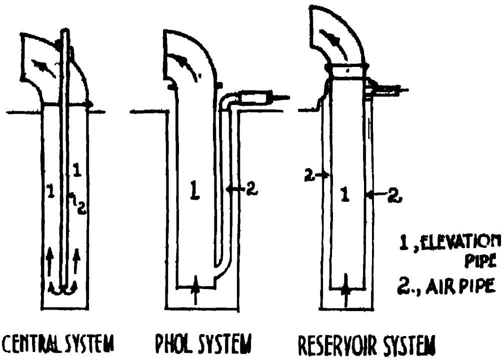


Fig. 45.

Air lift pumps are used to lift water from deep open wells up to 200 ft. and sometimes even 500 ft. maximum.

Formula for the volume of free air required is

$$V = \frac{L}{\log \frac{K\,(S + 34)}{34}}$$

Where, V = volume of air in c. ft. per gallon of water raised,

S = submergence in ft.,

L = lift in ft.,

K = constant ranging from 327 for low lift and high submergence, to 188 for high lift and low submergence.

Advantages of the air lift pump : (1) It does not require any priming like the centrifugal pump. (2) It can handle any quality of water (clear or muddy). (3) The space occupied by the air lift pump is very small, and it can be operated in locality. (4) There is no damage to the machine and consequent interference with the work, as there are no moving parts in water. (5) It can pump more water than any other similar machine. (6) A number of wells can be managed by one unit.

Disadvantages of the air lift pump : (1) The initial cost is high. (2) The flow is not continuous. (3) The efficiency of the pump is low. (4) The well must be made deeper for the proper submergence of the air-lift, and so extra cost is involved.

(2) Centrifugal Pump: This consists of a circular casing with an impeller inside it keyed on a shaft. The shaft is driven by power. Water is sucked in at the centre and is whirled out into a collecting chamber in which water goes on increasing gradually. (See Fig. 46)

CENTRIFUGAL PUMP

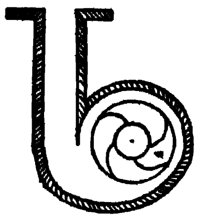


Fig. 46.

Single stage centrifugal pumps can be used for lifts up to 60 ft., but by placing appropriate number of stages side by side in the pump casing, centrifugal pumps can be used for lifts of even 1000 ft. The efficiency of an ordinary type varies from 55 to 75 per cent.

Centrifugal pumps are very commonly used for lifting water from tube-wells, canals and streams, because of their relatively low initial cost, simple operation, excellent durability, and high efficiency. They are particularly useful and suitable where large quantities of water have to be raised quickly.

A centrifugal pump can be worked by steam, using a portable or stationary engine. The fuel may be coal, charcoal, wood or even straw. An oil engine would, however, be not only easier to move and put up, but is more economical and simpler. The pump would discharge 1 cusec for 50 ft. depth and rather more for 35 to 40 ft. depth. The ordinary *mote* can only discharge 0·08 cusecs. Hence a centrifugal pump would be roughly equal to 12 motes.

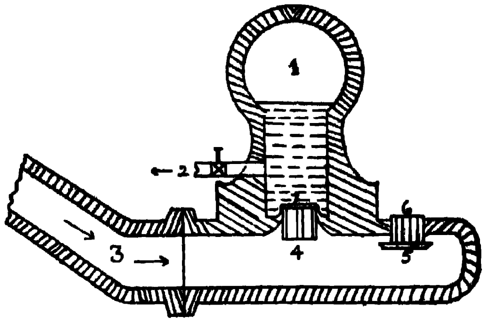
A centrifugal pump can be fixed above the water level or below it. When it is fixed above the water level, it should be primed before it can work, or else it should be self-priming.

(c) **14.** Sometimes in place of the mechanical power, **natural power** (i) water power (in the form of Hydraulic ram) or (ii) wind power (Wind mills) is used.

(i) **Hydraulic Ram**: Hydraulic ram (See Fig. 47) is a simple, automatic water-lifting device. It is an impulse pump. The power is obtained by the impulse of ram generated when a moving mass of water is suddenly stopped. Hydraulic rams are used, when a large flow of water at small head is available from a perennial stream, or a river or a tank, to pump a small quantity at high head. These are generally installed in the river bed where there is deep fall.

These are useful for lifting water only up to 125 ft. and are used for small irrigation works, because of their low running cost.

HYD. RAM



1. Air vessel. 2. Delivery pipe. 3. Water supply pipe. 4. Ram 5. Dash valve. 6. Opening. 7. Valve.

Fig. 47.

(ii) **Windmill:** The windmill utilises the natural power from wind. It consists of a big diameter vane wheel fixed on the top of a high steel or wooden trestle, and a plunger type reciprocating pump at its bottom. The upper end of the plunger rod is fixed eccentrically to the vane wheel. A suction pipe is fixed at the base of the plunger pump, with its other end dipping into water. A delivery pipe rises to a reservoir, fixed overhead permanently, where the water is stored, to be used whenever required. When a strong wind bows, it moves the vane wheel, which in turn works the pump and the water is lifted up. Vide Fig. 48.

WIND MILL

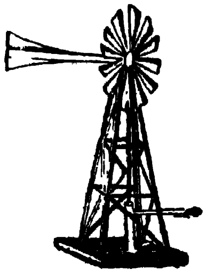


Fig. 48.

Aerometers: This appears to be the best kind of windmill. The efficiency of an aerometer is variable, but even the lowest may be taken at 10% whereas that of a good centrifugal pump is about 70% under effective and favourable conditions.

The H. P. obtained in the windmill varies with the areas of the fan and as the cube of the velocity of the wind. The discharge by a windmill is very nearly the same as for a typical *mote*. (0·08 cusecs).

The machinery of windmill which is found to work well in America is not altogether satisfactory in India. It is very light and frail, and frequently gets damaged or entirely collapses under a strong gale.

The atmospheric head for pumping though theoretically is 34 ft. from the lowest water level, yet only 16–20 ft. is allowed practically.

Diameter	Height of tower	Gallons lifted per hour	Remarks
6 ft.	25 ft.	2,000	Used for small head
18 ft.	46 ft.	20,000	For larger heads

N. B. Wind velocity is taken as 6 miles per hour.

Advantages: (1) Only installation charges are required, and the running charges are practically nil.

One need not depend upon imported material, like oil, etc.

Disadvantages: (1) It requires wind blowing at least with a velocity of 6 to 10 miles per hour, to make it work satisfactorily.

(2) It is not sometimes possible to work it, at the time when the supply is actually required. It is therefore necessary to have storage reservoirs along with it, which means considerable extra cost.

(3) Besides, the machinery generally gives trouble; as such the windmill is not generally used for irrigation works.

(4) The capacity of the mill should be large, if provision is to be made for windless days also.

(5) If there is no wind for a number of days, the windmill becomes useless.

It is only economical for irrigating costly garden crops where the lift is not much and the wind is favourable, and where a storage tank can also be constructed.

15. Advantages of Well Irrigation: (1) The main advantage in well irrigation is that the cultivator can raise water from the well, at his will and pleasure, and so he can arrange it to suit the wants of the crops.

(2) As it lowers the sub-soil water level, it is very effective in preventing water-logging.

(3) It is more economical for use, on account of its low cost and labour charges.

(4) As the cultivator can use the water whenever he likes, he can grow even two or three crops per year.

(5) The loss during transit is much reduced, as the water courses are generally well maintained (jungle or grass is cleared, and the surface of the distributary channel well consolidated or plastered or sometimes even lined).

(6) As the well water is used more economically than canal water, the duty under the well is naturally more.

(7) Well irrigation, can be used as a supplement to canal irrigation, when, in times of drought, there is not enough supply in the canal.

(8) As the supply in the well is generally constant, it can be used for intense irrigation of valuable crops.

(9) In cold weather, the well water is warmer, and in hot weather it is cooler; well water is therefore more agreeable to crops.

(10) The well may be sunk wherever desired.

(11) Economy of the well water is forced on the cultivator, as he has to spend money on lifting it, which is costly.

16. Disadvantages: (1) Well irrigation is usually costlier than flow or canal irrigation, being roughly 2 or 3 times costly.

(2) To lift the water, either manual labour or mechanical appliance is necessary. Both of these are liable to damage (sometimes). Hence water-supply may be interrupted and not continuous.

(3) The water in the canal brings fine silt and sand in suspension, which has good manurial properties, but the water in a well is clear and does not contain fertilising matter. Hence the land irrigated from a well is to be provided with manure now and then, which adds to the cost.

(4) The area under the well cannot be extensive, as the discharge from any well is generally small. That is to say, the area to be irrigated under a well can be only small.

TUBE WELLS

1. Introduction: In the ordinary or open well irrigation, the discharge that can be got from the well is very small, being 0·15 cusecs, so that tube wells were necessitated in order to get more

quantity of water to irrigate a large area of land. It is seen by experience that, on the average, about 1·5 cusecs is got from a tube well, and the duty realised is about 200 (acres) and a large area of 200 × 1·5 or 300 acres on an average can easily be irrigated by one tube well.

In a flat country like Uttar Pradesh, a number of such wells are sunk and for lifting water from these wells, power is necessary. The required power is generated in one place and distributed to all the wells. In India, first, Uttar Pradesh, and then, the Punjab, lead in tube well irrigation. It is said, that there are more than 2000 tube wells in U. P. and more than 1500 in the Punjab.

The biggest undertaking in India of tube well irrigation is the Ganga Canal Hydro-electric Scheme in U. P. In this project, it is proposed to operate 1500 tube wells in an area of 1300 sq. miles. The project seems to be a financial success.

In the Punjab, at Karol, 31 tube wells were sunk to irrigate 17389 acres, and it is seen that now about 29 wells are in good working order.

Construction: For the construction of a tube well, a tube is sunk into the ground, through water-bearing strata, and water from it is lifted by mechanical appliances (electric motor or oil engine).

TUBE WELL SHOWING DETAILS AT BOTTOM PORTION

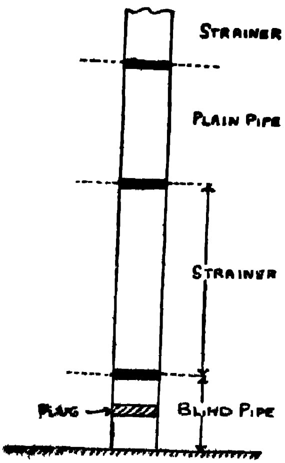


Fig. 49.

The tube well pipe consists of alternate lengths of plain and strainer pipes, (See Fig. 49).

The bottom-most portion of the tube well is a piece of plain pipe plugged at the bottom. It is also called *blind pipe*. This plugged bottom should be a little above the bottom of the bore; if not, it may give way under the weight of the pipe.

The plain pipe is a steel pipe with slots or perforation and the strainer pipe is a brass pipe. The plain pipe is provided in the impermeable stratum, and the strainer pipe throughout the

depth of permeable stratum. The strainer pipe is a perforated one, covered with a metallic net of very minute meshes. The water from the water bearing stratum passes first through the outward net and then through the perforations of the strainer into the tube. (See Figs. 49 and 52)

When the casing tubes are sunk for the full depths as required the inside tube is led down and shrouding is begun.

This process of shrouding is done very gradually, and as this proceeds, the casing is also withdrawn gradually.

Then a centrifugal or any other pump is installed in the

TUBE WELL SHOWING TOP PORTION OF PUMP AND PUMP HOUSE

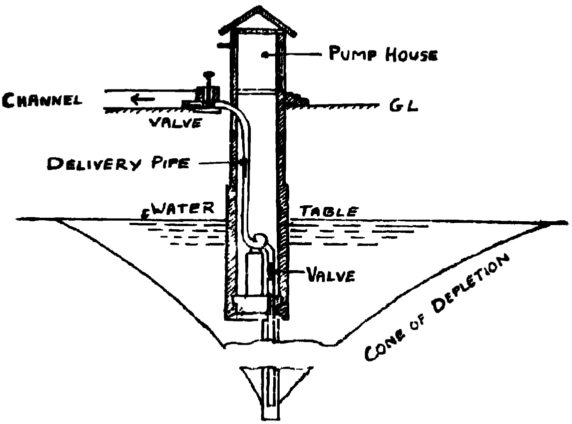


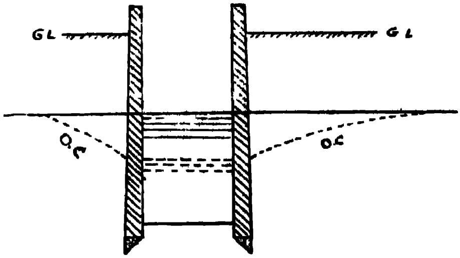
Fig. 50.

pump-sump, either above or below the water table, and the delivery pipe (from the pump) is fixed to discharge water into the field channel. A circular pump house of masonry is constructed round the bore. (See Fig. 50).

After the installation of the borewell, one attendent or *mistri* should be employed to note down the amount of water supplied to the cultivator, so that the proper assessment may be recovered from the cultivator; a mechanic should also be employed for the working and maintenance of the pump.

2. Some Definitions:—*Cone of depression or depletion:*

WELL



D. C.—Cone of Depletion

Fig. 51.

The water table at any place generally slopes towards the main drainage line. When a well is sunk and the water pumped out, even then, the ground water level remains undisturbed; but there is steepening of the slope towards the well, and this slope gradually increases as the pumping proceeds; and finally it assumes the form of a parabola and this parabolical surface thus generated, is called *the cone of depression* (See Fig. 51).

Depression Head :—This is the head or the vertical difference in height, between the normal groundwater level (before pumping) and the level of the water in the well, after pumping.

Aquifer :—Aquifer is a geological structure or a formation of the substratum, which is rich in ground water. The soils met with in the Ganga and Sindhu plains are good aquifers. These are well suited for the sinking of both open and tube-wells.

Specific Yield :—The specific yield of a well is the yield of the well in c. ft. per hour under a head of one foot. This is also called the *specific capacity*. It depends upon (i) the porosity of the aquifer, (ii) the amount of water in the well, and (iii) the rate of pumping.

Critical Velocity :—Owing to the cone of depression, the hydraulic gradient is steeper near the well and the velocity becomes greater; and as the depression in the well or the head increases,

the velocity becomes greater than what the material of the sub-soil can withstand. The velocity is called the *critical velocity* for the material, and it depends upon the head outside and inside the well.

Critical Head :—The head which induces the critical velocity is called *critical head.*

N. B. :—If the velocity exceeds the critical velocity, the soil particles will 'blow' into the well. The critical head for fine sand is from 5 to 7 ft. and the yield then is 16 gallons or 2·5 c. ft. per hour per sq. ft.

3. Necessity for tube well irrigation : In the Punjab and Uttar Pradesh, a large area of land could not be irrigated under the canal; hence some means had to be found for irrigating them, and as a result, the tube well irrigation came into use. The underground water was not being used at all, and so wherever canal water and surface water were not available, tube wells were sunk.

(b) Even in the irrigated tract, there was much water-logging. The sinking of the tube well lowered the water table and reduced the water-logging to a very good extent. In some places, it may be said, that the tube well irrigation has replaced canal irrigation altogether.

Thus the Government has saved a good lot of money which otherwise would have to be spent on remedial measures for water-logging.

4. Suitability for tube-well irrigation : The soil in the area to be irrigated should be surveyed by taking samples of soils in different portions of the area. Also the pH value should be tested (if it is between 7 and 9).

The position of the well should be located centrally, so that the length of any distributary is not greater than about a mile and any field is not more distant than about half a mile from the distributary.

The suitability of water for irrigation should also be tested and it should be seen that the salt index is negative.

The quantity of water available from the well, or the yield, as it is called, should not deteriorate in course of time.

5. Testing of a Tube Well : The complete test of a tube well after finishing the installation includes

(i) Discharge, (ii) Electric Consumption, (iii) Delivery and suction head (iv) Efficiency of the pump.

N. B. : For a depth of 12 ft., the usual discharge should be 1·5 cusecs. The yield of a well is affected by

(i) Choking of the strainer : (This can be generally managed).

(ii) Insufficient inflow to replace depletion : This insufficiency is due to the irregular presence of clay lenticles : (This cannot be managed).

6. Selection of Site for the Location of the Bore Well : There are various methods of locating a site for a tube well. They are

(i) *Water divining* : This is only prophesying, whether good and enough quantity of water is available at the site. This is only chance or luck, and not based on any scientific principles, though attempts are made recently by this method and some are successful.

(ii) *Studying the Symptoms on the surface of the ground* : The different symptoms are,

(a) The soil where water is available, is darker and damper than the surrounding places.

(b) The grass grown at this place is brighter than in other places.

(c) The trees also seem to grow better here than in the vicinity.

(iii) *Observing the geological strata* : The main test for the location is the geological strata at the place or site. Test bore holes should be taken and it should be seen whether below the soil crest, in the ground water reservoir, there is a coarse sand stratum, thick enough (even 100 to 150 ft.)

(iv) The sufficiency of water from the tube-well should also be tested and at the same time, salinity and the pH value should also be noted.

7. Types of Tube Wells : There are different varieties of tube wells in use.

They are,

(i) Strainer wells.

(ii) Cavity Tube Wells.

(iii) Artesian Tube Wells, etc.

The chief type is the strainer well commonly used and is described separately below.

8. Requisites of a Tube Well: (i) The velocity through the soil should always be less than the exit critical velocity.

(ii) The discharging velocity should be more than 3 ft. per second. This is to force out the sand in suspension from the water in the tube.

(iii) The area of perforation should be less than half the superficial area of the unperforated surface. This is to prevent eddies and backflows.

(iv) The straining material should allow maximum waterway

(v) The straining should not be in direct contact with the perforated tube, as there would be a likelihood of the perforations being choked up.

So an "ideal strainer" should be "an open hole to put down in an aquifer, which collects and allows to flow all the required water available in the aquifer, with no resistance to the flow or loss of head or collapsing and having unlimited life."

For this purpose, the size, number and shape of the screen-openings are of great importance; also to prevent collapsing and to impart unlimited life to the strainer, it is essential to make a proper choice of the strainer material.

The following metals and alloys have been found and they are developed to be corrosion-resistive and used for the manufacture of strainers.

(i) Zinc-free-brass or Cupro-nickel alloy

(ii) Stainless Steel.

(iii) Low-carbon Steel.

(iv) High-copper alloy.

9. Types of strainers used: There are different kinds of strainers in common use, but the best and the most generally used one, is the *Brownlie Strainer*. (See Fig. 52). It has the advantage of keeping the straining material away from the tube.

STRAINER (BROWNLIE)

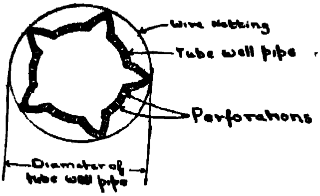


Fig. 52.

Another type commonly used is the *Phoenix Strainer*. This is plated with calcium, and so it is free from being choked by corrosion.

10. Choking up of Strainers: During the working of the tube well, the strainers generally get choked up in two ways:—

(i) Mechanical Choking and (ii) Chemical Choking.

(i) Mechanical choking is due to choking of the perforations or slits, with sand and silt. To guard against this, the velocity of inflow should be lower than the critical velocity; also this can be avoided by proper shrouding method. The pulsating action of a centrifugal pump is also useful in this respect.

(ii) Chemical choking is due to the presence of salt in water, especially Calcium Bicarbonate and also Sodium salts. This chemical choking is reduced to a good extent, by providing a large area of perforation in the tube.

N. B.: Strainer clampslip permanently removes the trouble of clogging.

11. Operations involved in the construction of a Bore Well:

(i) Boring to the required depth.

(ii) Lowering of the inner tube well pipe.

(iii) Shrouding.

(iv) Withdrawing the casing pipe.

(v) Construction of the pump-sump, well houses, etc.

12. Methods of Boring Tube Wells: There are different methods of boring tube wells and the most important or the usual types adopted are

(i) Percussion or Rope method.

(ii) Water-jet method.

(iii) Core-drill method.

(i) *Percussion Method*: This method is used in alluvial areas where the soil is mostly sand or gravel. A sludger is used for boring the hole which reduces the soil (in the strata) to powder, while boring.

(ii) *Water-Jet Method*: This method is used in hard clayey soil. By the use of the water-jet, the particles of soil are washed down, and the boring effected.

(iii) *Core-drill Method*: This is adopted when the stratum is hard rock. A drilling machine is used for the purpose and the power used is compressed air generally. The drill brings up the cores of hard rock to the surface, and they can be inspected easily.

13. Sinking the Bore Well: At the place where the tube well bore is to be located, a pit is excavated about 15 to 20 ft. deep and 6 to 8 ft. in diameter, and the tube of the required diameter with a cutting shoe* fixed to its bottom is lowered into it.

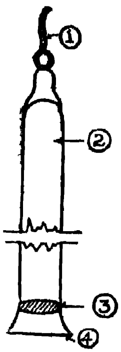
After lowering the tube into the pit, it is clamped into position with wooden clamps and partly filled with water (which is necessary for the boring operation). This is done until water is struck in the bore hole itself.

* *N. B.*—**Cutting Shoe**:—The cutting shoe is of two types, (i) *Slip Shoe* and (ii) *Screw Shoe*. These are of a slightly larger diameter than the boring tube, the object being to provide a clear passage for the bore, as it is being sunk. The cutting shoes are made of tempered steel. The slip shoe is larger in diameter than the screw shoe, and it is used in stiff clay, rocky soil and deep bores, as it can make a larger hole, and thus reduce friction during sinking and withdrawal of the bore tube. The screw shoe is screwed to the bottom of the tube and comes out when the tube is withdrawn. The screw shoe is used in sandy soil as this gives no extra clearance.

The slip shoe remains at the bottom of the well when the casing is withdrawn, but the screw shoe comes away with it. The shoes are of tempered steel and the inner tubes of the tube well are of ordinary steel.

The actual boring is done with a sludger or sand pump. This is 2 inches less in diameter than the casing pipe and its length is from 8 ft. to 10 ft. for boring from 8 inches to 16 inches diameter. A cutting shoe of tungsten steel is riveted to this sludger (at its lower end). This shoe is tapered at the bottom, for ease in cutting the material at its bottom. A flap or ball valve is fitted inside the sludger. (See Fig. 53).

SLUDGER



1. Rope
2. Pipe
3. Valve
4. Cutting Edge

Fig. 53.

A tripod is fixed at the site of the bore hole, from which a sludger is suspended by a wire rope from a pulley attached to the tripod. The leg of the tripod opposite the crab winch is firmly fixed in the ground, so that when there is a pull from the winch, the tripod may not turn over. The sludger is fixed centrally over the boring tube, and worked either by manual labour or by engine.

When the sludger moves down, the flap valve is forced open, and the loose material enters into the sludger pipe; when it moves up, the valve closes and the loose material is retained in the pipe. During the upward stroke, vacuum is created at the bottom of the bore and the pounded material in the bore is sucked up and gets mixed up with water at the bottom of the bore. This suspended material enters the pipe during the next downward stroke of the sludger. At the end of the downward stroke, the cutting shoe cuts some more material at the bottom, which is pulled up during subsequent strokes and thus the operation is continued for some time.

After about say 30 to 40 strokes, the sludger is generally filled with loose material, it is taken out and all the loose material emptied. The material is carefully examined for any indication of the change of the stratum and a careful record of it is kept.

At some convenient point above the ground level, the boring tube is clamped to the sleepers and over this, a platform is constructed for loading the weights, (a) to overcome the frictional resistance of the pipe, and (b) to force the boring pipe down as the

excavation by the sludger proceeds. As the boring progresses, the casing pipes are screwed one to the other, and lowered, till the required depth of bore is reached.

N. B. A difficult task in the tube well installation is the putting in of a water-tight joint between the top of the rising tube and the skeleton sluice valve. Recently this has been effectively managed by introducting a mechanical rubber stopper invented by Mr. A. M. R. Montague.

The excavated material got from the sludger indicates the type of stratum passed through. If it is water-bearing stratum, there is fine, medium or coarse sand gravel, or sometimes kankar. If it is clayey stratum, it is non-water-bearing.

When a water-bearing stratum is reached, water rushes into the tube. The level to which the water rises in the boring tube, indicates the pressure of water in the water-bearing stratum. This level should be observed in the morning before starting the work, and also during the progress of the work. The morning level gives the static pressure of water in the bore. The level taken during the progress of work in the day time shows the working pressure which is generally lower than the static pressure.

When the final depth is reached, the loading platform is removed and the clamps are attached to the strainer, which is then lowered into the casing pipes. The object of clamping is to support the strainer and to prevent it from falling into the bottom of the pipe.

One unit of the tube-well is screwed on to the strainer, and another clamp is attached to the newly screwed tube at a distance higher up.

The advantage of this method is that it is very simple; the disadvantage is the time wasted in removing the plunger sandpump everytime.

14. Discharge from a Tube Well: The formula for the discharge was first developed by Dupit in 1863.

Let Q = Total discharge of the well in cusecs.

D_1 = Depth of well below static water-table.

D_2 = Depth of water in the well when pumping.

R_1 = Radius of the circle of influence of the streams radially entering the strainer.

R_2 = Radius of the well.

p = Permeability.

DISCHARGE FROM A TUBE WELL

If x and y are the co-ordinates of any point on the cone of depression (See Fig. 54) then the cross-sectional area by a horizontal section through that point x through which the water passes towards the well is $2\pi xy$.

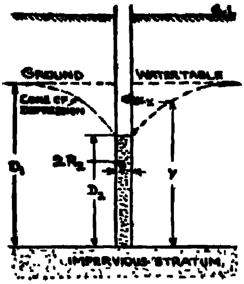


Fig. 54.

Solpe of water in the water–table

$$\text{underground} = \frac{dy}{dx}$$

According to Darcy's Law.

$$V = p\frac{dy}{dx}$$

$$\text{Hence } Q = p\frac{dy}{dx}\,2\pi xy$$

$$\text{or} \quad \frac{Q\,dx}{x} = 2\pi py\,dy$$

$$\int_{R_2}^{R_1}\frac{Q\,dx}{x} = \int_{D_2}^{D_1} 2\pi py\,dy$$

i. e. $$Q\log\frac{R_1}{R_2} = \pi p\left(D_1^2 - D_2^2\right)$$

Changing into common logarithms, we obtain for the discharge,

$$Q = \frac{\pi p\left(D_1^2 - D_2^2\right)}{2{\cdot}3\log_{10}\dfrac{R_1}{R_2}}$$

The following assumptions are made in the derivation of the above formula.

(i) Water table is at rest and is horizontal.

(ii) The bottom of the strainer reaches a horizontal impervious stratum.

(iii) The flow is radial and the discharge remains uniform along a series of concentric cylinders.

These assumptions are not realised perfectly and hence the formula can at best give only approximate results.

15. Comparison of the Discharge of a Tube Well with that of an Open Well : The discharge in a tube well is greater than that from an open well because,

(i) A greater velocity than the critical velocity can be allowed in a tube-well, as it is provided with a strainer.

(ii) The tube-well can tap more than one aquifer, except the topmost one, which if tapped will interfere with an open well. A plain pipe is, therefore, used in the first aquifer of a tube-well bore.

16. Causes of the failure of Tube-well irrigation: (a) The discharge of a tube-well may gradually deteriorate or become less and (b) in some cases, the number of tube-wells sunk for irrigating a certain area may not be sufficient.

Due to these reasons, failures are seen in the tube-well irrigation scheme.

17. Advantages: (1) The wells may be sunk at any place required by the cultivator.

(2) When the cultivator does not require water, he can immediately stop it, unlike in the canal irrigation, where the regulation is to be done by the concerned authorities. This improves the duty.

(3) As the assessment is based on the quantity of water used, the loss of water is much reduced.

(4) The losses in transmission are much reduced as the distribution channel is small in length and well maintained (and sometimes it is even lined with tiles or other impervious material).

(5) The supply may be made according to the requirements of the crop, and thus the maximum yield got.

(6) Isolated areas which cannot be managed by canal irrigation are easily managed by tube-well irrigation.

(7) It helps in lowering the water-table, and thus water-logging is reduced to a very good extent.

(8) When there is drought or failure of rain, underground water is used by the tube-well irrigation.

(9) It supplements canal irrigation in times of necessity, when there is a poor supply in the canal, either in summer or at the tail end of the season.

18. Disadvantages: (1) The lifting of water is by mechanical means and the cost on account of this, is high.

(2) Also, the mechanism may give way and affect the water-supply, and thus the growth of the crop. This failure of mechanism may be due to the failure of the energy supplied or of the working parts themselves. In either case, the water-supply is interrupted.

(2) The water from a tube-well is free from silt, and the lands will have to be manured, which means extra cost.

(3) The tube-well pipe may get choked up or corroded. This means additional time and cost for replacement.

19. Importance of Well Irrigation: In India, before partition, (excluding the states) the area under cultivation and irrigation was as follows;

In 1902—03 area sown = 22·6 crore acres.

Area irrigated = 4 4 crore acres, i. e., irrigated area works out to about 20% (nearly) of the total area sown or cultivated.

Details for the Area Irrigated

	Area		Percentage of irrigated area
By canals	1·7	crore acres	38%
By tanks	0·8	,,	18%
By wells	1·3	,,	29%
By other means	0·6	,,	14%
	4·4		

The statement above shows us the importance of wells in the Irrigation System of India, it being nearly more than 1/4; it may even be said that the area under wells is very nearly equal to the area under the canals.

The supply from the wells has an immense value in the years of drought, because tanks and canals are affected by the low rainfall.

The statement noted below shows the details of well irrigation in certain states of India, where well irrigation is on a large scale.

Provinces	No. of wells used for irrigation			Gross area irrigated in a normal year in acres	
	Permanent	Temporary	Total	Total	Average for well
The Punjab	275000	74000	349000	3750000	10·7
U. P.	500000	830000	1330000	5731000	4·3
Madras	626280	...	626000	2000000	3·2
Bombay	254000	...	254000	650000	2·6
M. P.	14000	42000	56000	77000	1·4

From the above statement, it is seen that out of the total of 13 millions acres which are irrigated by wells, $9\frac{1}{2}$ million acres lie in Upper India—The Punjab and U. P.—because of the fact that in the great alluvial tracts of North India, the sub-soil water supply appears to be practically inexhaustible. Also it may be noted that in Bombay and Madras, the average acreage per well is only (nearly) 1/4 that in the Punjab.

Questions.

INUNDATION CANALS

1. Describe how you would proceed to select the head for an inundation canal taking off from a meandering river in an alluvial plain.

How far down the approach channel would you locate the head regulator of the canal ?

(Bom. Univ. April 49 B. E.)

2. (*a*) What are inundation canals ? (*b*) In what type of country are they possible ? (*c*) What are the requirements of a good site for the head of such a canal ? (*d*) How are the bed level and F. S. L. of such a canal fixed ? (*e*) What considerations are made in the location of the head regulator of such a canal ? **(Poona Univ. Oct. 52, B. E.)**

3. Write short note on Inundation canal.

(A. M. I. E. Nov. 52. and Nov. 53)

4. Write an explanatory note on: Head of an inundation canal. **(Gujarat Univ. 53. S. E.)**

5. Write an explanatory note on: Inundation Irrigation.

(Gujarat Univ. 53. S. E.)

6. Write explanatory notes on: Inundation irrigation.

(Poona Univ. April 55)

BANDHARA

1. Draw a plan, elevation and cross-section of Bandhara. State the advantages and disadvantages of Bandhara Irrigation.

(Bom. Univ. Oct. 1949 B. E.)

2. What is a Bandhara? How is its site selected? Give the main features of Bandhara Irrigation.

(Gujarat Univ. 1953 S. E.)

3. (*a*) What are the favourable conditions for the selection of a suitable site for a Bandhara?

(*b*) Draw a plan and section of a needle Bandhara.

(Bom. Univ. 1953)

4. What do you understand by Bandhara Irrigation? What considerations go to determine the site of a Bandhara and the irrigated area established thereon?

(Poona Univ. April. 1954 B. E.)

5. Write Short notes on Bandhara Irrigation.

(Gujarat Univ. April 1954)

6. Describe Bandhara Irrigation and bring out its importance.

(Gujarat Univ. April 1955)

WELLS

1. What are the advantages of Tube Well Irrigation and in what circumstances can it be practised? Describe one method of raising water from a tube well, assuming your own data.

(Poona University, April 1955)

2. Write short note on: Chain pump.

(Gujarat Univ. April 1955)

3. Describe Artesian and Sub-artesian wells giving neat sketches.

(Gujarat Univ. April 1955)

4. Define the term "specific yield" of a well. How is it determined? Deduce an expression to determine the specific yield of a well in alluvial soil.

(Gujarat Univ. April 1954)

5. Describe with necessary mathematical expressions the Recuperative Test for testing the yields from (i) open wells and (ii) tube wells sunk in alluvium.

(Bombay Univ. 1953)

6. Explain with sketches the construction of a tube well. Under what conditions is tube well irrigation possible? What are the advantages of this type of irrigation over the ordinary canal irrigation?

(Poona Univ. 1953)

7. Write an explanatory note on: Critical exit gradient.

(Poona Univ. November 1953)

8. Write short note on: Specific yield of a well.

(Poona Univ. November 1953)

9. What are the usual methods of observing discharges in tube wells?

(A. M. I. E. November 1952 and 1953)

10. Differentiate between shallow, deep and artesian wells.

Describe briefly the construction of a tube well in alluvium.

(A. M. I. E. November 1953)

11. What are the various methods of lifting irrigation water from open wells and tube wells? Draw sketches and describe briefly one most common method for each kind of well. Write a note on the degree of permissible salinity of irrigation water from wells.

(Gujarat Univ. April 1953)

12. What are the sources of underground water supplies for irrigation? Write a note on tube well irrigation. Give a typical cross section of a tube well.

(Gujarat Univ. S. E. 1953)

13. Write explanatory note on: (1) Karez (2) Air Lift Pump.

(Gujarat Univ. S. E. 1953)

14. Write a short note on: Artesian well.

(A. M. I. E. November 1952)

15. Write a short note on: Critical Head for a well.

(Poona Univ. October 1952)

16. Write a short note on: Windmill Irrigation.

(Bombay Univ. April 1950)

17. What are the different methods of testing yields of tube wells and under what conditions are they used ?

What is understood by critical velocity of the inflow in a *tube well and what are the consequences of exceeding this velocity?*

Sketch the vertical section through a tube well in alluvial and sandy soil showing clearly the subsoil water level, the cone of infiltration and the different parts of the tube well.

(Bombay Univ. April 1950)

18. Under what conditions is tube well irrigation feasible? What are the advantages and disadvantages of this type of irrigation ? *Draw a sketch of a tube well.*

(Bombay Univ. October 1949)

19. On what considerations is the head to an inundation canal from an alluvial river selected and what steps should be taken to minimise the entry of silt in such a canal ?

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

20. Write short notes on
   (1) specific capacity of wells.
   (2) silt factor.

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

21. What is inundation irrigation ? How does it differ from other types of irrigation ? Explain with a sketch the principle of water distribution in inundation canals.

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

22. Write short notes on Specific capacity of wells.

(Bombay Univ. B. E. Civil (New Exam.) November 1954)

23. Write short notes, with special reference to the situation necessitating their use or application, on
   (1) Bhandara Irrigation.
   (2) Block System.

(Bombay Univ. B. E. Civil (New Exam.) April 1955)

24. Compare the discharging capacities of manually operated and animal-driven lifting devices and the conditions under which each can work best.

(Bombay Univ. B. E. Civil (New Exam.) April 1955)

25. Describe recuperation test. Why is it preferable to pumping test ?

(Bombay Univ. B. E. Civil (New Exam.) April 1955)

26. A sugarcane plot 4 acres in area is proposed to be irrigated by digging an open well. The water bearing strata consist of coarse sand and the permissible depression head is ten feet. Pumps will work for 12 hours per day. Find the size of the well required.

(Bombay Univ. B. E. Civil (New Exam.) April 1955)

27. Write explanatory notes on Bhandara Irrigation.

(Bombay Univ. B. E. Civil April 1956)

28. Explain why one tube well of 6″ size bored in sandy or alluvial soil can yield more water than 3 open wells of 10 ft. diameter.

(Bombay Univ. B. E. Civil April 1956)

29. Describe any one method of sinking a tube well in alluvial formation. What methods are adopted for increasing the discharge from a tube well or to make it more dependable in its working.

(Bombay Univ. B. E. Civil. October 1956)

30. How is specific capacity of an open well in alluvial soil determined.

(Bombay Univ. B. E. Civil. October 1956)

31. Write explanatory notes on Phad System of Irrigation.

(Gujarat Univ. S. E. Civil. April 1956)

32. (a) What are artesian and sub-artesian conditions ?

(b) Explain by sketches, the various circumstances under which such conditions are possible.

(c) How will you estimate the yield from a shallow or deep well ? What physical observations are necessary for this purpose ?

Problems : For problems and their solutions please refer to pages 15 to 17 of the book "Solutions of Problems in Irrigation Engineering" *by Shahane and Iyengar.*

CHAPTER VIII

STORAGE RESERVOIRS

1. Storage Reservoirs: A storage reservoir (or tank) for irrigation is formed by throwing an embankment or dam across a natural water-course or nalla or river, and the water collected on the upper side of this is drawn by means of sluices in the dam, through the channels which supply water to the lands.

2. Necessity for Storage Reservoirs : Works for the storage of water are necessary in many parts of India.

(1) When in an area, the usual rainfall is not enough for the crops, water is stored in reservoirs and allowed to lands whenever necessary.

(2) In some areas, the rainfall may be confined to certain parts of the year, and even here water will have to be first stored and then distributed to the lands during the other periods of the year.

(3) In places like Baluchistan, and Rajaputana, where the streams flow like torrents for only a few days in the year, storage is a necessity to ensure proper water-supply to the crops.

3. Surface Tanks : One of the earliest systems of irrigation in India, where the rainfall, though copious and sufficient, was badly distributed during the season, was by surface-tanks.

Such tanks are most numerous in Madras where millions of acres of rice crop are irrigated by them : also in the old Mysore, and Dharwar and Kanara of the old Bombay State.

These tanks vary in size from a few acres to 9–10 square miles of water surface. They are formed by throwing earthen embankments across small streams. The surplus from the tanks flows into those below. Sometimes these bunds get breached from heavy rainfall, and in bad years the tanks fail (to fill.)

The Madras and Mysore tanks depend mainly on the rain in their own catchment, which is often very small; consequently they are sometimes fed from rivers or streams, by means of channels called feeder channels.

For the construction of tanks the embankments are of different sizes. In the hilly country they will be deep, and in the plains they will be of small height.

4 Requirements of a Storage Reservoir: An idea reservoir should satisfy the following conditions ;

I. It should have a channel bringing down an ample supply of water.

2. There should be a broad expanse of nearly level ground in front of the embankment, or dam, to form the bed of the reservoir, having a slight dip towards the bund.

3. The land to the rear or the downstream side of the bund should be of much greater extent than the bed, and slightly lower in its level, in order that every portion of it may be commanded by the tank and irrigated by the sluice in the embankment, from which one or two channels take off and lead the water to the fields.

4. Rock or other foundation, impervious to water, should be met at only a small (reasonable) depth from the surface.

5. Stone, fuel, lime and other materials required for construction should be available within a reasonable distance for a masonry dam, and good suitable earth, as well as stones for pitching, for an earthern dam.

6. The soil for the construction of an earthen dam for the reservoir should be of retentive nature.

7. Valuable garden lands or wells or village sites should not be submerged under the reservoir contour.

8. The site selected should give the required storage with the shortest length of dam.

N. B. Although the sites which give the shortest length of dam are the most favourable ones, yet they will have sometimes to be rejected, in favour of another, which though giving a longer length, will have the advantage of holding up the largest quantity of water.

As the depth of water depends on the height of the embankment and the slope of the bed of the reservoir, and the storage depends upon the area of the water spread and its depth, it is found, in certain cases, that a longer bund may give a larger area and greater depth and thus provide a greater storage which will amply repay the cost of the increased length of bund.

9. The site should be favourable to locate the waste weir, preferably in a saddle, so as to pass off all the flood water into the natural drainage stream without artificial ones and protect the embankment.

It rarely happens that all the conditions noted above, are met with simultaneously; but the site which offers the most of these advantages should be selected; it being remembered that the reservoir is essentially intended for irrigating the lands below it; hence the selection of the site must pre-eminently depend on considerations of the command available for the lands to be irrigated.

N. B.: Irrigation by storage reservoirs or tanks is often combined with that of rivers, the water from the rivers being brought into the tanks (that are favorably situated) by means of feeder channels from the neighbouring streams.

5. Feeder Channels: These may be classified under two categories.

(1) Those which intercept only the catchment area of the ridge above the feeder channel.

(2) Those which draw water from a weir or an anicut, constructed across the stream. Vide Fig. 55.

FEEDER CHANNEL

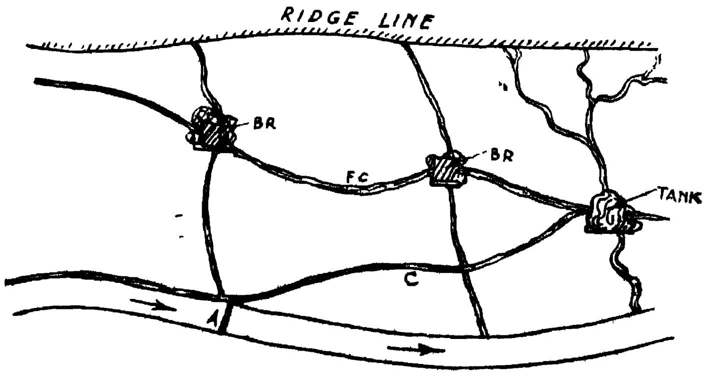


Fig. 55.

A – Anikat or Weir. C – Channel from Anikat. F. C. – Feeder Channel. B. R. – Balancing Reservoir. T. – Tank.

The bulk of the rainfall is generally got in very short periods having heavy rainfall; hence the special design for the feeder channel.

Design of Feeder Channel: If the channel section is small, it becomes insufficient during heavy rains. If the section is made too big, it will be prohibitively costly. Also during low flood, there is heavy loss due to absorption, evaporation, etc. Hence a compromise between the two types, as noted above, is made, while designing the section of a feeder channel. The section of the channel should be made big enough to take one-eighth of the ordinary heavy run-off or one-fourth of the yield to the reservoir. Generally the capacity required to be stored in the reservoir is arranged to be got in 3 to 4 days' flow in the channel.

Sometimes, balancing reservoirs or auxiliary reservoirs are constructed on the way, where cross drainages meet the feeder channels. Vide Fig. 55. By adopting this, not only a good quantity of silt is prevented from flowing into the channel, but breaches in the channel embankments are also minimised.

Special points to be noted in the design: (a) A feeder channel should enter the reservoir with its bed level at F. R. L. of the reservoir.

(b) There should be no direct-irrigation under the feeder channel on its way.

The feeder channel may be constructed so as to divert water from a separate valley, by constructing a channel from this valley and crossing the ridge separating the two valleys, by a deep cut or Tunnel. (See Fig. 56). The water from here may be going to waste or there may be no people to utilise it. Also this valley may be getting more rainfall and in all these cases, the diversion of water from one valley to another, through a feeder channel is advantageous, though somewhat costly initially. For further particulars on feeder channels please refer to authors' book "Solutions to Problems on Irrigation Engineering" Pages 87 to 89.

FEEDER–CHANNEL

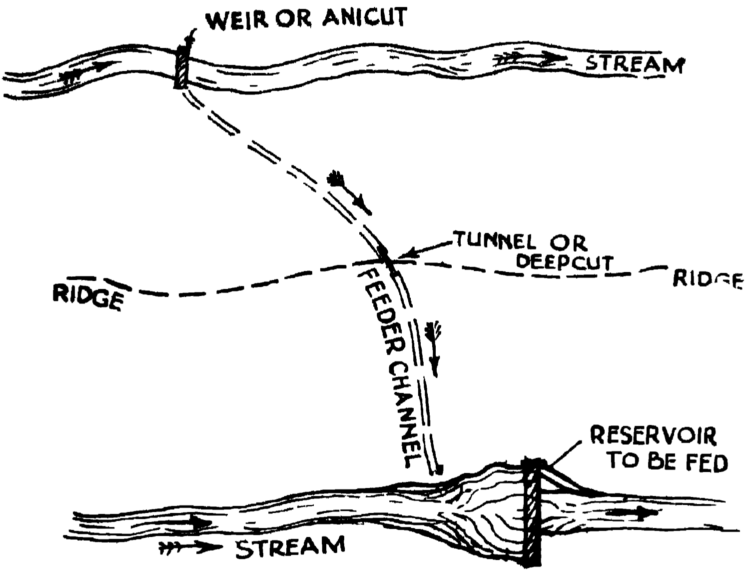


Fig. 56.

6. Supply from the Catchment Basin: A register should be maintained showing the water level of the tank for the whole year. When the water is above Full Tank Level, the actual discharge over the weir should be recorded; also the quantity of water used when the water is below F. T. L. should be noted. Thus the total quantity of water received into the tank, or the yield from the catchment basin in one year, and thus for a number of years, or the average receipt, should be calculated and recorded.

7. Storage Capacity: The command under any tank is usually ample and the storage capacity should be restricted to that which can be replenished in an ordinary year, as it is always advisable that the irrigated area under a tank should not fluctuate considerably. A slight increase in capacity is permissible and necessary.

If the waste weir site demands the rising of the crest by the reduction of cutting for the waste weir and channels, to provide for future silting of the tank and allow a good margin for any

reduction in capacity, this increase should be only to a small extent; that is about 10 %, (more than this, will not be economical), because if the capacity is increased too much, the tank will fill up in good years only and in other years, part of the capital cost will remain unproductive.

To determine this capacity, the following survey details should be got:

1. A contour at the level of the sill of the lowest sluice.
2. A contour at the full tank level.
3. A sufficient number of cross sections, between these two contours, to admit of laying down contours at intervals of 1 ft. vertical.

The cross sections should be taken in directions which would be at right angles to each other. (N. S. and E. W.)

It is generally advisable to prolong the level of the lines of cross section for 3 – 6 ft. above the F. T. L. to settle questions relating to the increase of the capacity, to effect the tank as a Flood Moderator.

8. Calculation of Storage Capacity : The calculation for the storage in a tank is made by adding up the extent between each of its contours.

The general practice is to take contours at vertical distances of 10 ft. and interpolate others at 1 ft. interval.

From the gross capacity of the tank, the storage below the sluice sill level must be deducted, and the result is the capacity of the tank that can be used for irrigation; and again from this, a further reduction is to be made for loss of storage due to

(a) Evaporation, *(b)* Absorption, and *(c)* Percolation in bed, and the balance left over is to be taken as the useful capacity for irrigation or the '*net available capacity*' as it is usually called.

Different Formulae : (to find the capacity).

(a) If only two sections (contour areas) are taken

$$V = \frac{d}{3} \left(A_1 + A_2 + \sqrt{A_1 A_2} \right)$$

(*b*) If there be three equidistant horizontal sections, the contents are, $V = \frac{d}{3} (A_1 + 4 A_2 + A_3)$

(*c*) If there is an even number of equidistant horizontal sections A_1, A_2, etc., up to A_n at a common distance *d*, the capacity or volume

$$V = d (1/2 A_1 + A_2 + A_3 + \ldots\ldots + A_{n-1} + A_n)$$

where V is the volume or capacity in c. ft.

d depth or height between the horizontal sections in ft.

A_1, A_2, A_3......A_n are the areas of horizontal sections in sq. ft.

9. Mass Curves : Mass curves are used to work out the capacity of reservoirs. These are obtained by plotting the cumulative outflow, and the cumulative storage into the reservoirs, against time. The inflow is corrected for evaporation and other losses. The inflow, the evaporation, irrigation requirements, the discharge over the waste weir are all obtained for successive weeks or months (over a period of a year) and from these, the cumulative totals calculated and plotted against the corresponding period of time.

Fig. 57 shows a typical mass curve diagram. OQ is the inflow

MASS CURVE DIAGRAM

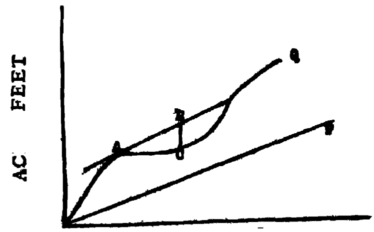


Fig. 57.

and OP is the demand. The demand is the sum of the demands from the reservoir for irrigation, hydro-power, and sluicing purposes. Suppose A represents the beginning of the longest dry period on record, if a line is drawn parallel to OP and tangential to A, then the ordinate BC will represent the storage necessary to maintain a rate of inflow not less than that represented by the slope of the line OP.

Fig. 58 shows the mass curves for the inflow and total outflow. Since the total outflow is equal to the total inflow, the inflow and outflow curves begin and end at identical points. The maximum vertical distance is the storage required to meet the needs of the project.

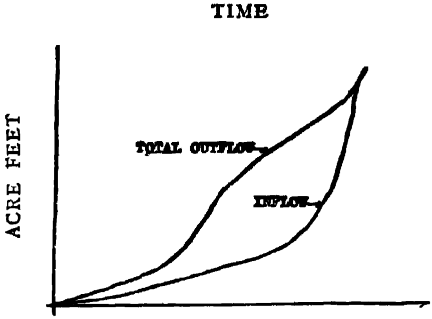


Fig. 58.

The present tendency is to use a variation of the above mass curve, by taking the average inflow for the period on record as zero line. This curve can also be used in the ordinary manner, to find out the required storage, or rates of draw off from the reservoir. In this case, to determine the true rate of use, the slope of the rate lines must be added to the average flow.

10. Determining the Height of the Embankment (or Dam for Storage): Surveys are taken and the capacity for storage at different levels of the proposed reservoir is calculated for intervals of 1 ft. vertical.

The catchment area of the reservoir is known and the run-off also is either gauged or fixed or assumed from neighbouring similar catchments. Again the average bad year's rainfall and the run-off are also got. Thus the quantity of water that can be stored in an average bad year is calculated. As the sill level of the sluice is fixed at some height over the bed of the river, the storage required for irrigation will be only above this level. From the Tables available, the level at which this quantity is available, can be found out, and this fixes the F. T. L.

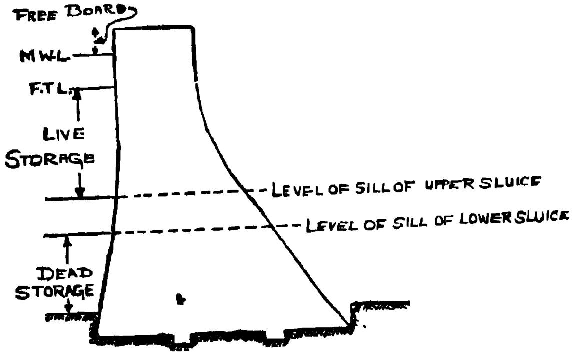
In the rainy season during heavy rain, there will be heavy flood in the stream and the water level in the reservoir will rise to

a certain height, above the crest level of the reservoir. This height depends upon the length of the weir provided, and other local circumstances which should be settled beforehand.

Even during heavy floods, there may be high waves acting in the lakes and these may overtop the dam. To protect against this, and for other casualties, a good margin of height called Free Board varying from 3 ft. to 10 ft. is provided.

The height of the dam or embankment for a storage reservoir should, therefore, provide for all the above items. See Fig. 59.

DAM SECTION SHOWING SUPPLY IN RESERVOIR (CAPACITY).



Sketch showing Height of Dam and Free Board.
Fig. 59.

The height from the bed level of the river to the top of embankment or dam at any point is called the Height of the Dam.

N. B. When fixing the height of an earthen dam, provision should be made for settlement also.

11. Selection of Site of a (dam) Storage Reservoir:

In choosing a site for a reservoir, it should be remembered that it is the longitudinal section of the site that generally determines the cost of the scheme. The site available for waste weir determines the feasibility of the scheme, and the nature of the tank basin, determines the relative cost of the storage.

The best site available for a dam is usually the one which has the ridges running down from the highland on both sides to the stream to be impounded; such ridges will greatly reduce the cost,

as the section of the dam varies roughly as 2½ times the square of its height. Low long dams are thus frequently cheaper than short high ones, and they are also much safer. A ridge however offers feasibility for the proper drainage of the dam; but it should not be very narrow; for if it is so, this may render it liable to leak, and may not allow space for the future raising of the embankment, should that become necessary, and will also tend towards heavy cost and difficulty in construction. Generally the best site is a gorge where part of the dam is separated from the flank embankment, by a hillock rising above the level of the top of the dam. The two portions of the dam can be completed independent of each other. This formation is peculiarly suited for a composite dam, as the gorge portion can be constructed in masonry, while the flanks can be constructed in embankment. Vide Fig. 60.

TYPICAL SITE FOR A DAM

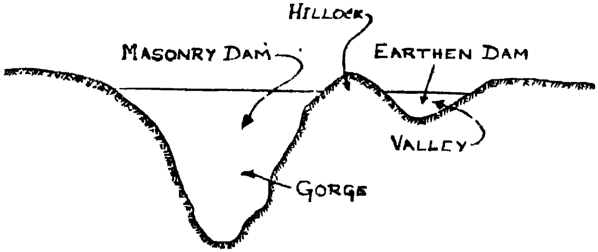


Fig. 60.

A good dam site is one that gives the maximum storage for the minimum length.

One of the most favourable sites for a dam is below the junction of two streams, as then, the storage will be obtained up both the valleys; such a place is frequently an indication of good foundation, as hard substratum may have deflected the tributary into the main river; and also of the ridge lines of the opposite sides of the valley approaching each other.

The longitudinal slope of the river within the tank basin varies in the Bombay State from 2½ to 60 ft. per mile. In respect of the storage capacity, a slope less than 10 ft. per mile may be considered a good one; one from 10 to 20 ft. a fair one, and one 20 ft. per mile is a poor one. These slopes can be modified by

means of the ridge lines, that is, the nature of side slope, and the existence of the tributary basin.

12. Definitions: *Full Tank Level F. T. L. or Full Lake Level*: When water in a reservior is stored up to the crest weir in the reservoir, it is said to be Full Tank Level or Full Lake Level or full reservoir level F. R. L.

High Flood Level or Maximum Water Level: During the rainy season, especially during heavy rain, the discharge in the stream is at its maximum level. This is called High Flood Level H. F. L. Or Maximum Water Level M. W. L.

Free Board: When the reservoir level has come up to the high flood level, to prevent the flood water overtopping the dam or embankment, by the action of waves, it is necessary to allow a margin of some height above the High Flood Level. This is also necessary to allow for any settlement or any inequalities at the top surface of the dam or embankment due to the neglect of the ryots (for maintenance) or natural causes. This height of free board is usually between 3 to 10 ft. according to height of the dam and the action of the waves.

Gross Storage Capacity: The total quantity of water that can be stored in a reservoir up to the crest of the weir from the very bed of the reservoir is called the Gross Storage Capacity.

Available Storage Capacity or Live Storage Capacity: Every lake or reservoir is intended to supply water to the lands below it. For this purpose, sluices are provided with their sills at certain levels.

The water above this sill level only can be made available for Irrigation. And the quantity of water so available from the lowest level of the lowest sluice to the crest level of the weir is called Available Storage or Live Storage.

Dead storage: This is the storage of the lake or tank below the sill level of the lowest sluice. As this cannot be used for irrigation, it is called dead storage.

Net Storage or Effective Storage Capacity: All the water available between the sill level of the lowest sluice and the crest of the weir cannot be made available for irrigation as a portion of it is lost in evaporation and absorption. The remaining quantity only

can be made available for irrigation, and this is called Net Storage or Effective Storage Capacity.

Valley Storage :—Valley storage is the volume occupied by a stream or river, when it is in flood, after it has overflowed its banks.

13. Choice of the type of dam :—

(1) *Site* : If the site of a dam is in a gorge which is very narrow, an arch dam is convenient, if the foundation is good i. e., rocky.

(2) *Nature of the Foundation* : If the foundation for the seat of a dam consists of ordinary soil (earth or soft rock) it is advantageous to have an earthen dam, especially when the height is not much.

If there is good depth of soil overlying the rock, even then, an earthen dam is cheaper than a Gravity or Buttress dam.

If sound rock is available, a masonry or concrete dam is advisable, provided all the materials are available nearby.

(3) *Discharge of the catchment* : If the weir to be provided is large, then a gravity type of dam is better.

If the diversion work of the river is heavy and tedious, a Masonry or concrete dam is preferred.

(4) *Communication* : If communication between the two sides of the river is very essential and there is no bridge nearby, either an earthen dam or Gravity type of dam, with enough breadth at top for the roadway, is recommended.

(5) *Availability of Materials* : If the materials required for either an earthen dam or a Gravity type of dam are available cheaply and at a reasonable distance, one of this type is preferred; otherwise a Buttress dam is advised.

N. B. If the storage is only for temporary use, and the height of the dam required is small, a Timber dam is preferred, if the required timber is available nearby and in plenty.

(6) *Climate* : Where the rainfall is heavy, it is advisable to construct the dam with Masonry, as it can withstand the downpour (better than an earthen dam).

(7) *General* : If the structure is to be permanent and to avoid any risk, especially in the case of water-supply storage, a Masonry dam is preferred.

Dams : A dam is an impregnable and impervious barrier thrown across a natural drainage line, to impound water (up to a certain limiting level which is usually lower than the top of the dam,) on its upstream side. Its main function is to store water either for irrigation or water-supply or to produce power.

14. Classifications of Dams : Dams are usually classed as (1) Earthen dams and (2) Masonry Dams; but logically they may be divided as (1) Non-rigid dams and (2) Rigid dams.

(1) Non-rigid dams : These dams are subdivided into (a) Earthen dams and (b) Rock-fill dams,

(2) Rigid Dams : Rigid Dams are subdivided into (a) Gravity Dams, (b) Arched Dams (c) Arched-Buttress Dams, (d) Reinforced Concrete Panel and Buttress Dams.

N. B. Of these, the first two are primary and the last two are of a combination.

Rigid dams are constructed not only of masonry and concrete but also (on some occasions) of steel or timber.

Non-rigid dams are mostly of earth and so they are permeable to a certain extent.

Earthen dams may be subdivided into (a) Dry-fill dams and (b) Hydraulic-fill dams. The Dry-fill dams are further subdivided into (a) earthen dams with core walls and (b) earthen dams without core walls, and the Hydraulic-fill dams into (a) fully hydraulic, and (b) semi hydraulic.

Thus the classification of Non-rigid dams may be given in a tabular form as noted below.

- Non-rigid dams
  - Earthen Dams
    - Dry fill Dams
      - Dams with core wall
      - Dams without core wall
    - Hydraulic fill Dams
      - Fully Hydraulic
      - Semi Hydraulic
  - Rock fill Dams

In addition to the above, there are other types of earthen dams known as compound dams and composite dams which will be described separately.

N. B.: Some people classify the Rock-fill dam as a semi-rigid dam. Summarising the above, dams may be classified as,

Rigid Dams:

(a) Masonry Dam
(b) Arched Dam
(c) Arched Buttress Dam
(d) Reinforced concrete-panel & buttress dam

$\Big\}$ These will be dealt with in chapters X and XI.

(e) Steel Dam.
(f) Timber Dam.

Non-rigid Dams:

(a) Earthen Dams, (These will be dealt with in chapter IX)
(b) Hydraulic fill dam.
(c) Rock-fill dam.

15. Steel Dams: These are not generally much in use. Even in America, where they are introduced, there are very few of them and it is said that there is none in India.

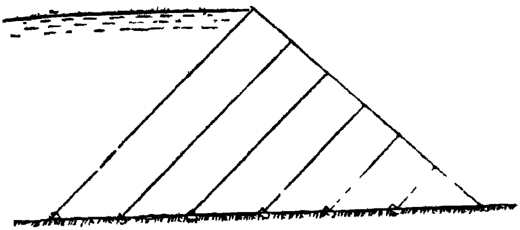
In principle, steel dams are ordinary buttress dams, and the usual height up to which they are built is about 50 ft.

The Buttresses are built 8 ft. apart centre. Inclined girders are fixed on the upstream side of the buttresses and to the flanges of these girders, steel plates are riveted to hold the water in front. A concrete wall is constructed at the bottom of the girder in the river bed, and the lower edges of the plates are embedded in the concrete wall, so that the dam may be water-tight.

Steel dams require a foundation of sound rock for anchoring.

Types: Two types of the steel dams are in use: (i) The direct strutted steel dam and (ii) The cantilever type steel dam.

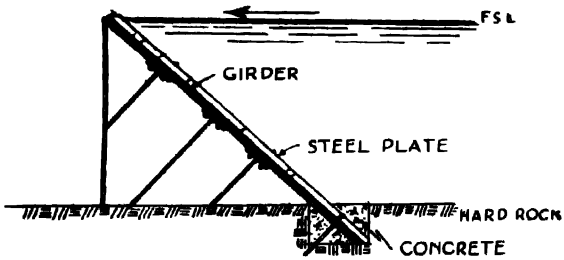
(i) *Direct Strutted Type*: This is the simplest of the two. Here the load is carried directly from the deck to the foundation through the inclined struts (See Fig. 61).



STEEL DAM
Direct-strutted Type.
Fig. 61.

(ii) *Cantilever Type*: This type is more common. Here the buttress-members are mostly oblique to the deck girder which should be properly anchored in the river bed.

A sketch of this type is given in Fig. 62.



STEEL DAM
(Cantilever Type)
Fig. 62.

N. B.—Anchorage for taking up tension, and for restraining temperature movements at the abutments and the foundation, can be got by drilling anchor plain bars into the rock and grouting.

Advantages

1. It requires the least width of base.
2. It is more adaptable to spillway type dams, as it has a vertical downstream face.

When satisfactory anchorage cannot be got, the direct-strutted type should be used.

Steel dam requires a foundation of sound rock for anchoring.

Advantages of steel dams, in general.

1. It is easy for inspection.
2. It is convenient for future extension.

3. It can be maintained foi a long period with proper attention and frequent painting. (With the development of noncorrosive steel, the need for painting may be lessened or even eliminated).

4. It is cheap in the long run.

5. The construction is rapid.

6. It is suitable for any ordinary width of river.

7. It resists unequal settlement due to flexibility.

8. It can be easily repaired, in the case of a leak, by the modern welding processes.

9. The stresses in the dam are more determinate.

10. The structure is not affected by frost action.

Disadvantages:

1. Special precautions will have to be taken, to preserve the structure to resist any temperature effects.

2. It is not as rigid as a concrete dam.

3. It requires anchoring to foundation.

4. Being light, it cannot absorb shock, when water overflows.

N. B. Steel arch dams may be feasible, but it is seen that no dam of the type is built anywhere.

16. Timber Dams. Timber dams are employed where timber is available in plenty and cheap and the height of the dam is only small (say about 30 feet) and the work is also temporary.

Disadvantages:

1. Maintenance charges are heavy, especially where heavy floods and ice-runs are frequent.

2. Leakage is also heavy and this is often difficult of repair, especially when the dam is high.

3. The life of a timber dam is short.

These disadvantages have created a prejudice against timber dams.

Classification: Timber dams are classified as:

1. A-frame type dams,
2. Beaver type dams.
3. Rock-filled crib type dams.

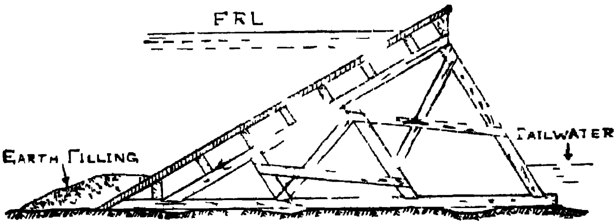
(1) A-frame type dam. This type is best suited to earthen foundations. It is built of square timber and planks. The deck is generally made 30° or less with the horizontal, See Fig. 63.

The stability depends upon

(a) The weight of water on the deck,

(b) The anchorage of the sills to the foundation.

TIMBER DAM



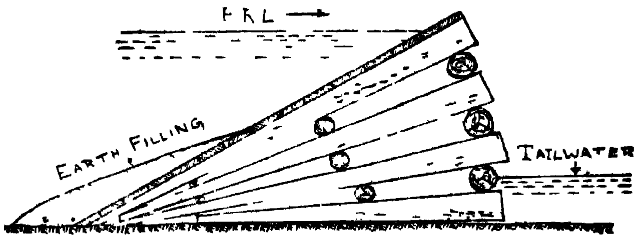
A-Frame Type.
Fig. 63.

Advantage. (1) It costs less (2) Maintenance charges are small.

Disadvantages. It fails when neglected.

(2) Beaver Type Dam. This type is used for low heads. For the bents round timber is used. The slope on the water side should be made as flat as 2 to 1, See Fig. 64. The ends of the

TIMBER DAM



Beaver Type,
Fig. 64.

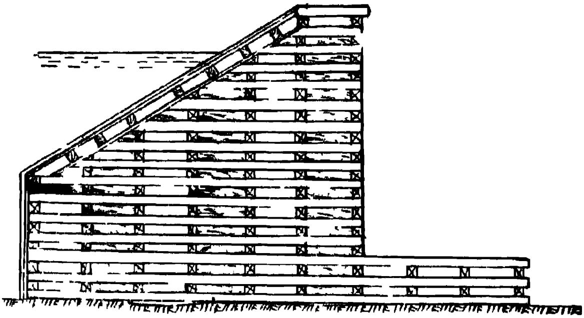
timber towards the upstream are drift pinned together, and the bottom timbers are fastened with anchor bolts to the foundation. The deck is either of planks (or branches of trees—sometimes).

Advantages:

1. Lowest in cost, if timber is available.
2. Lower maintenance charges.

(3) Rock-fill crib type. See Fig. 65. In this type, cribs of round or square timber are bolted together and filled with rock and boulders, and a plank deck is placed on the top. The bottom of cribs should be anchored to the rock foundation.

TIMBER DAM



Rock-Fill Crib Type.
Fig. 65.

When the dam is intended for overflow, it should be protected against erosion, when the foundation is soft. This is done by sloping or stepping the downstream face, and providing an apron to protect the foundation. This apron is anchored to piles. At the lower end of the apron, big boulders are paved to the bed to prevent undermining, See Fig. 66.

CRIB TYPE OVERFLOW DAM

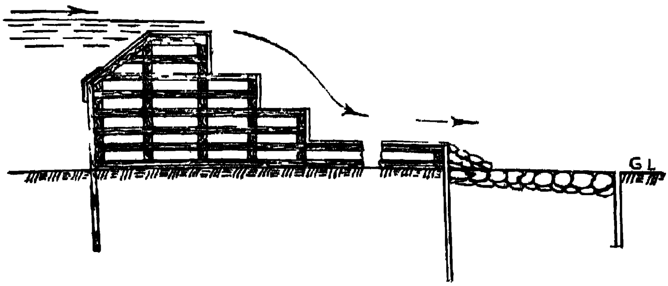


Fig. 66.

Advantages. As it is supported by rock fill, it will stand for some time even after the timber decays.

Disadvantage. Repairs to this type is difficult.

17. Rock-Fill Dams :—These are dams of loose rock. They were first used by miners in the western parts of U. S. A. where the cost of cement was prohibitive. Most of these structures are located in deep canyons, as it was easy to convey stones by gravity to these sites.

Where used :—Rock-fill dams are suitable

(1) Where the required quantity of good rock for the rock-fill can be had at the dam site itself.

(2) Where suitable materials for an earthen dam cannot be got.

(3) Where the cost of cement for the construction of a *pucca* masonry dam is high.

Foundation :—The foundation of a rock-fill dam need not be hard rock. It is enough, if it can just take the weight of the superstructure above, and if it can only stand the erosion due to percolation of water (a soft soil is not fit for this purpose, but an ordinary hard soil or soft rock is enough).

Design of the section :—The crest width usually adopted varies from 5 to 30 ft. according to requirements, and the height of the dam. (as noted below).

Height of Dam	Under 100 ft.	100–150 ft.	150–200 ft.	Greater than 200 ft.
Crest width	8 ft.	10 ft.	12 ft.	15 ft.

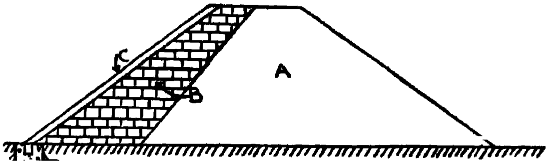
Sometimes, an additional width is given when a highway is required to pass over it.

The front upstream slope should approximate the angle of repose. The usual slope given is noted below.

Height of Dam	Less than 50 ft.	50–100 ft.	100–150 ft.	Greater than 150 ft.
Front slope	$\frac{1}{2}$ to 1	$\frac{3}{4}$ to 1	1 to 1	1·3 to 1

The rear slope varies from 1·2/1 to 1·4/1. The flatter slope is given in the lower levels.

SECTION OF ROCK–FILL DAM



A–Rock–Fill B–Dry Rubble or Masonry section
C–Impervious Membrane D–Cut–off wall.

Fig. 67.

The section of a rock-fill dam usually consists of

(a) The rock-fill.

(b) The dry rubble or masonry section in front.

(c) The impervious membrane just at the water surface of the dam (in front of the masonry section).

(d) The cut-off wall. (See Fig. 67)

(*a*) *The Rock–Fill*: The rock-fill is the main part of the dam and the stones required should be sound. When the sides are steep and high, the stones required for the fill are shot down. On large works, they are transported by dumptrucks on rail-road using side-dump cars.

(*b*) *Dry rubble or masonry section*: The function of this is to support the membrane in front, and retain the loose fill at its rear, and at the same time, to transfer the water load to the rock-fill.

The stones used in this portion should be of sound rock, properly shaped and of good size, and should be properly bedded; good workmanship is required for this purpose. When done well, the percentage for voids varies from 25 to 32 only. The thickness of the masonry section depends upon the slope and height of the dam, and also on the type of the equipment used. This thickness increases downwards. (The thickness at the top is from 5 to 10 ft. and for every 100 ft. height this thickness is increased by 5 ft.)

(c) *Impervious Membrane*: This should be as watertight as possible. It is jointed to a cut-off wall at its lower end, by this flexible membrane.

The membrane may be of any material, namely,

(1) Concrete (Gunite and R. C. C.)

(2) Timber

(3) Steel.

(1) *Concrete Membrane*: This may be of three different types, viz.,

(i) Monolithic, (ii) Sliding, and (iii) Laminated.

Of this, the one in common use is the monolithic type. Here the concrete is poured against the masonry section, to form the watertight membrane. This may be either rigid or flexible. The rigid type is used where the dam is not high. The flexible type is the one mostly used.

Gunite facing: This is more dense and impervious than concrete. This is used in low and moderately high dams.

(2) *Timber facing*: This was being used in dams constructed in olden days where timber was available in plenty, and now they are not much in use. The advantages claimed for the facing, were, (1) it was flexible, and (2) there was no damage to the structure during settlement. (3) It was used below ground water level and, to preserve the timber, it was being impregnated with creosote.

(3) *Steel facing*: The steel used is copper-bearing steel. This is to reduce corrosion. The plates used vary from 1/4 inch to 1/2 inch in thickness. They are anchored to the rubble backing behind. The expansion joints used are of the U type. These are protected by bituminous paints and are found successful in many places.

(d) *Cut-off wall*: To prevent water stagnating behind the face membrane, and also to prevent the uplift pressure, a cut-off wall is provided. A trench is excavated in the hard rock bed, the width of which being as per requirements and the depth depending upon length of the path of percolation. The trench is filled with concrete into which the membrane is fixed.

N. B.—The length of percolation provided in solid rock or impervious foundation is usually 10 to 15 per cent of the height of the dam.

Settlement of Rock-Fill Dam: The rock-fill generally settles, as it contains a good amount of voids. To reduce this settlement, water is jetted into the rock-fill as the work progresses, and it is continued until the construction is over. This process allows a good settlement even to begin with and also increases the stability of the dam.

N. B. The vertical settlement is roughly 5 per cent of the height.

Failure of Rock-fill Type: (i) A rock-fill dam fails on account of insufficient spillway.

(ii) Slopes steeper than the angle of repose, are also a reason for the failure of the rock-fill dam.

In India the rock-fill dam is considered dangerous to construct, in view of its apparently temporary nature.

Recently, a development of the rock-fill dam is made in types—composite type and core wall type—as noted below.

18. Composite type of rock-fill dam: A cross-section of this type is given in Fig. 68. It is very stable and the struc-

ROCK–FILL DAM. COMPOSITE TYPE

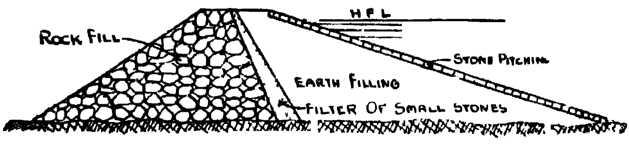


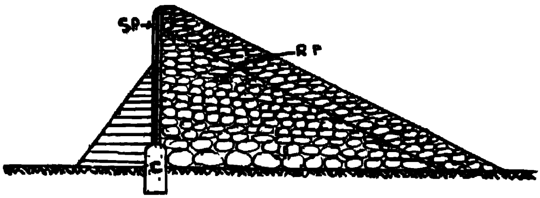
Fig. 68.

ture is safe. The rock-fill is on the downstream side and the earth-fill on the upstream side; and it is protected by stone pitching. The earth provides the water-tightness required of a dam. In between these two sections a thin layer or filter of small stones is provided. The downstream portion provides the required drainage and adds stability to the structure. The filter is an essential necessity, as the fine material should not easily pass away to the rear.

19. Core wall type of Rock-fill Dam: In this type, a core is fixed in the centre of the dam. This core may be either of metal or masonry or earthwork.

(a) A *metallic core* is generally a steel diaphragm with a thin protecting layer of asphalt concrete of 4″ thickness on each side of it, (See Fig. 69). This is the type of section used for the rock-fill type dam, at Utah, U. S. A.

ROCK-FILL DAM WITH STEEL PLATE CORE.



C—CONCRETE WALL ON HARD BED ROCK.

S. P.—STEEL PLATE CORE WITH ASPHALT CONCRETE 4″ THICK ON EACH SIDE OF THE PLATE. R. F.—ROCK-FILL.

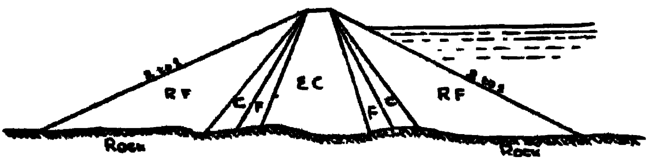
Fig. 69.

(b) *A core of masonry or concrete* is also sometimes used. This serves the same purpose as a metallic core and is better suited.

(c) The third one is the type with an *earthen core* in the centre of the rock-fill dam.

This type has less seepage than the type with the concrete face only. The core should be protected on both its sides, as usual, with a layer of filter (See Fig. No. 70).

ROCK-FILLED DAM WITH AN EARTHEN CORE.



R F.—ROCK-FILL. C—COARSEN MATERIAL.
F.—FINE MATERIAL. E. C. EARTHEN CORE.

Fig. 70.

One special point to be noted in rock-fill dam is that the surplus discharge should never be allowed to pass over the dam, or even to rise to this level (top of dam). The weir should be located in a side valley nearby. Also the outlet of the irrigation sluice should be located very carefully. It should be separate from the dam proper, or if at the dam site itself, it should be placed in a tunnel or a deep trench, cut in the solid rock bed of the dam.

20. Hydraulic-Fill Dam : This is more common in America than elsewhere. This type is suited, where the dam is situated with high banks for the rivers and where the required material for the construction is available in plenty.

(1) Materials : The material required should contain more of gravel and sandy particles (the materials with a high percentage of impervious stuff is not suitable). The soil should be well graded, the material ranging from gravel to silt or clay. The percentage of clay and silt should not be more than 15 to 20 per cent.

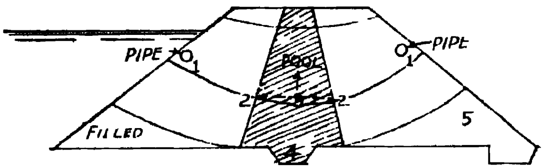
(2) Construction : On either flank of the dam proper, high level reservoirs are constructed, with pipes fixed at their bottom portion. These are fitted at their ends with large size nozzles, (called giants or Monitors) and these nozzles are directed to the bank (containing the required material for the dam) to undercut the material (the velocity produced in these monitors is of the magnitude of 100 to 200 ft. per second). This operation makes the mixture a *slurry*, which is arranged to flow through pipes or flumes to the site of the dam.

When the material issues out of the tube in a liquid form, the coarser material is dropped at the outer edge of the embankment; the fine material moves towards the centre, the finer still further, and the finest to the centre of the dam itself (which is called the core portion), where it is precipitated forming a wide clayey core-wall. This method is called *sluicing or Hydraulicing.* Thus the outer section of the dam is more pervious than the inner sections and facilitates drainage. In this operation, the material is transported by water from the source to the site of working, and even here, the distribution work in the horizontal section of the dam is done by water alone.

The fine materials, namely, silt and clay, as they get deposited in the centre of the dam, form a watertight core.

As seen in Fig. 71 a pool is formed at the centre and the extra

HYDRAULIC FILL DAM



1 COARSE MATERIAL. 2. FINE MATERIAL
3. VERY FINE MATERIAL 4. CORE. 5 DRY PLACED MATERIAL.

Fig. 71.

water is accumulated at the top. This extra water is now and then pumped out and the core becomes gradually stiff. The core should be made narrow; if not, it will exert pressure at the sides and produce slides in some cases.

Sometimes the materials are first excavated and brought to a convenient place, high enough to transport them through the pipes in liquid form, and water is then led into this place, through these pipes, and the whole mixed and converted into slurry and then conveyed to the site through pipes or flumes. The recent method of construction is as follows. The bottom portion or section of the Hydraulic Fill type is first built, as in the usual Dry Roll Filling Type and then the Hydraulic method is adopted (See Fig. 71).

(3) Requirements: In this type, an abundance of water, high enough above the top of the dam to permit of the flow of the material through the pipe or the flume, is required. Also, there should be a good supply of suitable material as said above, and it should be conveniently situated at each flank. Hence this type cannot be constructed under all conditions.

(4) Advantages: (i) Where possible to construct it, it is found to be cheap, and (ii) it is rapid in construction.

(5) Disadvantages *or Objections to its use are*

(i) The layers are stratified.

(ii) They are deposited in a very wet condition.

(iii) The method cannot be adopted universally in India (as water cannot be made available in all places, at a sufficient elevation).

21. Semi-Hydraulic Fill Dam : This type is used where the materials available are of uniform gravelly material and deficient in fines. This requires good underdrainage.

In this type, the materials are not transported by water. They are brought to the site by other means and dumped there, and only distribution of material at the site into their final positions is done by water. When the water jet is used to distribute the materials from the faces of the dam to the interior, the materials at the edges remain impervious and the materials at the centre become more pervious and contain a good amount of water. This exerts a large hydraulic pressure and as a result slides generally occur.

This method is cheap and convenient. But the difficulty is the maintenance of the wide pool in the centre, which is troublesome and may cause slides; special precautions should be taken to prevent slide by providing proper under-drainage.

22. Conditions favourable for the location and construction of an earthen dam : *An earthen dam may be located*

(i) Where in the foundation no rock is available, but only good compact soil is found. (Sodden earth and pure sand should be avoided).

(ii) Where the height of the dam is not much (say 70 to 100 ft. maximum).

(iii) Where the initial cost of construction is to be kept low.

(iv) Where the period of construction and the completion of the work is not fixed, and the work might be done slowly.

(v) Where good stone, lime, etc., required for Masonry dam are not available, but good earth and gravel can be got nearby.

(vi) Where ordinary unskilled labour is available for the work.

23. Conditions favourable for the location and construction of a Masonry dam : *A Masonry dam may be located*

(1) Where good hard rock is available at a reasonable depth.

(2) Where a suitable site for the location of surplus discharge cannot be got.

(3) Where the dam is very high, and the foundation can withstand the stress on it.

(4) Where the rainfall is heavy so that the structure can withstand the downpour.

(5) Where silting up of the reservoir is to be avoided. (This is done by the provision of the required scouring sluices).

(6) Where the site has got steep side slopes. (An earthen dam is likely to slip here).

(7) Where the period of construction and completion of the work is fixed and the work has to be done rapidly.

(8) Where the risk of failure cannot be allowed; as for example a water-supply storage reservoir for a big city.

(9) Where the required materials and skilled labour are available nearby.

24. Earthen dams : *Advantages of earthen dams* :

(1) They do not require expensive and solid foundations.

(2) The height of an Earthen dam can be raised from time to time to increase the storage capacity or to make good deficiency of storage due to silting up of reservoirs, and it lends itself to future improvements.

(3) The construction of an earthen dam is rapid and can be done with unskilled labour and with the materials available on the spot.

(4) When the materials required for the construction are available nearby, this becomes the cheapest type of dam.

(5) It also helps during famine relief, as ordinary labour required for the work can be made use of.

Disadvantages of earthen dams :—

(1) Floods are likely to damage the dam, as it is exposed to them, unless special precautions are taken.

(2) Weirs may have to be constructed at a heavy cost, as water should not be allowed to flow over the top (of an earthen bund).

(3) The earthen dam is not of a permanent nature (but it can be maintained for a long time with proper care).

(4) It requires constant maintenance and repairs, especially the slopes and drainage, which means extra cost.

25. Masonry Dam:

Advantages:

(1) The area of land required for the construction of the (Masonry) dam proper is less than in the case of an Earthen dam, and hence the compensation amount is much reduced.

(2) Though the initial cost of construction of a Masonry dam is high, the subsequent cost of maintenance is low.

(3) The failure of a Masonry dam is always preceded by a warning and hence the damage incurred is small.

(4) As scouring sluices can be conveniently located in a Masonry dam, there is less trouble due to the silting up of the reservoir.

(5) In a Masonry dam, the flood can be made to discharge over a lower portion of the dam or through the under-sluices which is not possible in an Earthen dam.

(6) The Masonry dam can be constructed to any height (The Boulder Dam in America is 726·4 ft. and the Bhakra Nangal Dam in the Punjab-India is 670 ft.) whereas the height of an Earthen dam is generally limited to about 80–100 ft. maximum.

(7) The Masonry dam is more permanent and reliable,

Disadvantages: (1) The cost of construction of a Masonry dam is usually high.

(2) Unless the foundation is hard and rocky, a Masonry dam cannot be constructed.

(3) Skilled labour and good materials, such as stones, lime, fuel, machinery, etc., are required for the work.

26. Comparison of a Masonry dam with an Earthen dam:—

Masonry Dam	*Earthen Dam*
(1) For the foundation, good rock should be available at a reasonable depth.	No rock is required for the foundation; any ordinary soil, except pure sand and sodden soil, is enough.
(2) The side slope of the site of the dam should be steep.	The side slopes of the site of the dam should be flat, otherwise the banks will slip.
(3) The dam may be constructed to any height, on solid rock, which can take the pressure over it.	The height of an Earthen dam is generally restricted to about 60–70 ft. (even up to 100 ft.)
(4) Skilled labour, and materials, such as stones and lime, should be available nearby.	No skilled labour is necessary, and any cheap Woddar labour is enough; only enough quantity of earth and stones for pitching should be available.
(5) The progress on the work is rapid.	The progress on the work is slow.
(6) Initial cost is high, but maintenance charges are low.	Initial cost is low, but maintenance charges are high.
(7) A number of large undersluices can be provided to pass off the early flood discharge and thus reduce the silting up of the reservoir.	Scouring sluices cannot be constructed conveniently as in a Masonry dam, and the reservoir is soon silted up.
(8) Any failure of a Masonry dam does not generally involve much disaster to life or property.	The damage done to an earthen bund is usually great, and sometimes involves loss of property and lives also.
(9) Where rainfall is heavy Masonry dams are suitable.	The earth available may not be able to withstand the heavy rainfall.
(10) Masonry dams are more reliable, as only a small number of accidents have occurred.	Earthen dams, unless done very carefully, are not safe.
(11) For any future raising of the dam, a Masonry dam is not very economical.	An Earthen dam lends itself easily (economically) to future improvements.

27. Flood Absorbing Capacity of Reservoirs: *Reservoirs are naturally good moderators.* During the monsoons, a good amount of water is absorbed in the catchment and stored and during the dry season, this is gradually let out by percolation throughout the whole year, enabling the river to maintain its dry weather flow also.

Flood Absorbing Capacity of a Reservoir: At the beginning of high floods, the flood water or the surface of the reservoir rises slowly, because the depth of discharge over the weir is small and the quantity of discharge is small, and in consequence, the flood waters are kept in the reservoir for some time. This action of the reservoir is called *flood moderation* or *flood absorptive capacity* of the reservoir. It is also called *flood storage.*

As the flood increases, and thus the depth of discharge over the waste weir also increases, the quantity of discharge increases. But after sometime, the incoming flood decreases and the depth of discharge increases and the storage becomes less and less; that is the surface of the reservoir falls down gradually.

Hence the reservoir moderates the intensity of the flood by absorbing part of it when it enters the reservoir, and discharging it after some time, that is, after the flood has diminished.

To take the best advantage of this property of flood-absorption, the capacity of the reservoir between the full reservoir level and the high flood level should be made large, compared with the inflow from the catchment area.

This property of flood absorption is made use of in preventing a flood in a river, damaging the country all round, and for this purpose, flood retaining basins, etc. are constructed.

28. Practical Method of Using Flood Absorption Capacity of a Reservoir: Sluices are provided at low levels in reservoirs and during the first floods, these are kept open and the flood is discharged through them. When next heavy floods come, they have first to fill the reservoir and then rise in level, and hence during this period, the rate of discharge from the reservoir will be less than the actual high flood discharge. So even when there is an intense flood of short duration, it is converted into one of less volume and long period.

Even in big reservoirs, the level of high flood is generally restricted to 6 ft. so that there may not be any danger to the tail channel, when there is a greater discharge over the weir.

Generally no allowance is made for flood absorption when high flood level is less than 6 ft. The length of waste weirs can be reduced to even 50 per cent, if allowance for flood absorption is made. This is done only for large works having a good length of surplus works.

For designing the length of the waste weir, taking into account the flood absorptive capacity of the reservoir, the following formulae are adopted: (The following formulæ and the table are taken from Bombay Specifications).

Having chosen a height of flood h, we have

$$h = \frac{QT}{12894\ A} = \frac{QT}{12900\ A} \text{ (roughly)} \quad \ldots \quad (1)$$

and getting h from this formula, L is found from the formula

$$L = \left(Q - \frac{7744\ Ah}{T} \right) \frac{1}{1 \cdot 33\ h^{3/2}} \quad \ldots \quad \ldots \quad (2)$$

where Q is the maximum discharge from the catchment area,

T is the time taken in hours for the reservoir to rise from full-supply level to high-flood level,

A is the mean area of water spread in sq. mile,

L is the length of the water in ft., and

h is the distance between high flood level and full-supply level, i. e., height of flood.

The time taken for the calculations adopted in parts of Uttar Pradesh is given below in a tabular form:

Area in sq. miles	Duration in hours
0–5	2
5–15	3
15–30	4
30–50	5
50–100	6
100–200	6–9
200–300	9–12
300–400	12–15
400–500	15–18
500–1000	18–21
Over 1000	24

Questions

1. What are the chief factors to be considered in the selection of site of a reservoir ? Explain with sketches, how far these are satisfied either at Bhakra or Mettur or at any other storage system under construction or completed in India.

(A. M. I. E., May 1951)

2. Explain the following terms as used in irrigation practice: Hydraulic fill and Rock-fill dams.

(A. M. I. E., November 51 and 52)

3. Explain ' Valley Storage. "

(Bombay Univ. B. E. Oct. 1952)

4. (*a*) When is the necessity for a reservoir felt ? Discuss it in detail.

(b) What considerations would you keep in view while designing a reservoir ?

(c) How would you calculate the quantity of water to be stored in a reservoir ?

(A. M. I. E., May 1953)

5. Explain the various considerations involved in fixing the height of a dam. What do you understand by the "Optimum storage level of a reservoir" ? How is that evaluated ?

(Poona University, November 1953 B. E.)

6. In what particular situation and in what general circumstance would you prefer, for irrigation storage, an earthen dam to a Masonry dam ? What considerations are involved in arriving at a suitable design of dam ?

(Poona University, April 1954)

7. Describe the construction of a Hydraulic fill dam ? Under what conditions would you adopt this type of construction ?

(Poona University, April 1954)

8. Write short notes on :

(i) Rock-fill dams.

(ii) Losses from reservoirs.

(Gujarat University, 1954)

9. Distinguish clearly between (i) Hydraulic fill type, (ii) Dry fill type earthen dams.

(Gujarat University 1954)

10. Write short notes on : (1) Rock-fill dams, (2) Hydraulic–Fill dams, (3) Flood absorption Capacity of reservoir.

(Gujarat University, 1955)

11. What are the points that guide you in the selection of site for a dam or a reservoir ? How do you choose the materials of construction for a dam ? **(Gujarat 55)**

12. Write explanatory notes on Rock-fill dams.

(Poona and Gujarat University, 1952)

13. In what circumstances do you consider a storage reservoir scheme suitable ? State the various investigations necessary and the favourable results to be expected for a satisfactory scheme.

(Poona 55)

14. Write short notes on Mass Curves.

(Bom. Univ. B. E. Civil April 1954 & 1956)

15. Write short notes on Hydraulic Fill Dams.

(Bom. Univ. B. E. Civil, Nov. 1954)

16. Write short notes on with special reference to the situation necessitating the use or application of : A Rock Fill Dam.

(Bom. Univ. B. E. Civil April 1955)
(Guj. Univ. B. E. Civil April 1956)

17. (a) Explain the different purposes that a storage reservoir serves and describe its various appurtenant works for the control and regulation of water.

(b) How do you determine the requirements of a dam for a perennial irrigation scheme? Illustrate the same with the help of suitable diagrams.

(c) What are the usual losses from such reservoirs? What steps are taken to reduce the losses or how are they allowed for in the design ? **(Bom. Univ. B. E. Civil April 1956)**

18. Discuss the various aspects to be taken into account for adopting one of the following as Head works for irrigation purposes.

(1) Storage Reservoir
(2) Solid Weir
(3) Barrage.

Suppose there is 2,00,000 of acres to be irrigated in the South Satara district. Which of the above works will you choose to construct across the river Krishna and why ?

(Poona Univ. B. E. Civil April 1957)

Problems: For problems and their solutions, please refer to page 12 to 14 of the book " Solutions of Problems in Irrigation Engineering " by the Authors.

CHAPTER IX

EARTHEN DAMS

1. Theory of Earthen Dams: An Earthen Dam resists the pressure of water with its huge mass without any scientific disposition of the material being possible, for it takes up so much space that numerous cases arise, where a solid structure could be built of minimum bulk. Besides, there is a limit to the height of an earthen dam (say 100 ft. generally). In the case of an earthen dam, the water pressure acts prejudicially, due to infiltration, thus diminishing the frictional resistance and adhesion of earth. The frictional resistance is measured by the angle of repose (The constants for it, for different soils will have to be determined. It is more for coarse soil, and less for fine soil). The adhesion or cohesion may be measured by the depth to which an unsupported piece of earth-work will stand. This latter property is very varying and uncertain being increased by a moderate amount of moisture and diminished by excessive wetness. Cohesion of clay is largely increased by a small amount of moisture, and this also increases the coefficient of friction, but an excess of it acts as a lubricant, in diminishing it.

The whole mass is treated as homogeneous from top to base and a uniform slope is adopted throughout, though the lower portion of a high dam is naturally in a different condition from the upper portion, as the former is subject to the weight of the super-incumbent material and is therefore more compressed: besides, the increase of moisture at the base diminishes the frictional resistance and cohesion. Further the variation in the nature of materials, their position and methods of construction, introduce several other forces which cannot be theorised.

From experience and observations, it is seen that, in a homogeneous dam with plain slopes, the resistance to slipping decreases with the height from the top. Proper section for it is the one having the slopes continuously flattened towards the base.

Low dams can be constructed with much steeper slopes than high ones, and although it was, at one time, thought that water

faces of dams require a flatter slope than the rear ones, experiments on the saturation of earth-work on dams have shown that flatter slope is necessary on the rear side to keep the earth-work above the line of saturation.

The stability of earth therefore depends on the ease and rapidity with which it can be drained of all superfluous water, because earth which is much charged with water is very unstable, while only slight damp earth is quite stable. Besides, it is not possible to take into account, in the design of earthen dams, the modification in the behaviour of earth-work due to causes operating subsequent to its construction which affect the lower parts more than the upper, and thus destroy the homogeneous nature of the mass which is presupposed for purposes of design. Hence in the design of earthen dams, the necessary data should be obtained by observation of existing work under similar conditions and similar strata, and supplemented by experiments. These experiments should be regarding the nature of soil to be dealt with, and for ascertaining the line of saturation in existing earthen dams of various designs.

2. Foundation: An Earthen Dam can be constructed on almost any foundation, and in any case, it should be roughly explored and the design adapted to fit in with the construction.

For example: (1) If the foundation is of plastic clay, it should be very carefully tested and all possible action taken regarding the safety of the structure. Here the slopes should be made, as flat as possible, to keep the stresses of the foundation sufficiently less than the strength of the material.

(2) If fine sand is got in the foundation and if its density is less than the critical density, then it will flow like a liquid, when saturated and loaded, if there is any disturbance; hence it should be marked, and properly compacted before any new earth is put in.

(3) Coarse sand and gravel are good for foundation. They give no trouble, they consolidate under the load and are stable.

(4) Rock foundations are sometimes got. In this case, proper grouting should be done, before the construction of the new embankment.

Generally the main point to be considered here is the final cost, and whichever is more economical, should be adopted.

3. Materials: The materials commonly used for an earthen dam are either adhesive or non-adhesive. The adhesive soils are clay, silt and fine sand and the non-adhesive soils are gravel and sand.

Properties of soils are mentioned in Chapter III. The adhesive materials become powder when dry, and with a little moisture become campact, and with more moisture become slushy, and when dry the surface cracks; the soil is generally impermeable. The non-adhesive soils—sand and gravel—are more permeable, and there is no adhesion between the particles. They are stable, and good at resisting friction and shear; as such, either of these materials cannot safely be used independently for an earthen bund. Hence suitable materials should be selected. Hence, with sand and gravel, enough clayey material should be added (one clay to one shale) to bind them together so that they may become watertight. To the clay and silt, enough shaly stuff should be added, to give the required frictional resistance from slipping and to be self-draining.

4. Requisites of a good soil for the dam :

(i) It should not be found charged with sand in large proportion.

(ii) It should not become slushy when wet.

(iii) It should be easily compressible.

(iv) It should not break with sharp angles and shining surfaces.

N. B.—Peat should not be used for an earthen dam. Pure clay by itself is not suitable. Pure fine sand has been used successfully in many places.

The usual tests made in connection with the soils are

(i) Mechanical analysis.

(ii) Permeability.

(iii) Voids ratio.

(iv) Density.

(v) Shear.

(vi) Presence of soluble materials, etc.

The additional tests to be made for adhesive soils (materials) are

(i) The Consolidation test.

(ii) Optimum moisture content.
(iii) Shrinking limit.
(iv) Expansion.
(v) Plastic limit.
(vi) Liquid limit, etc.

5. Ideal soil: " The correct conception of an *ideal soil* for best soil stabilization is that the soil should like concrete, contain the required quantity of coarse material, its filler and its binder. The coarse material in the form of sand provides internal friction and hardness. The filler, in the form of silt, provides embedment for the sand grains, and the binder, in the form of clay, coats the surface of silt and sand and provides cohesion. There is natural stickiness in the clay and this property is further augmented by surface tension of the film of moisture covering the various particles. Ideal proportions are,

Clay	...	5 to 10 per cent.
Silt	...	10 to 20 per cent.
Sand	...	70 to 85 per cent.

6. Stability of Earthen Dams: A dam is said to be stable if the result of all forces acting on the dam does not allow the dam to move.

The sum of all the forces resisting the movement should be far greater than the result which tends to move the dam i. e., the factor of safety should be great, say even 5. For stability in Earthen Dams the following items are generally considered.

(i) Stability against Head Water Pressure. This is more important in a masonry structure than in an Earthen Dam.

(ii) Horizontal shear in Downstream portion.
(iii) Horizontal shear in Upstream portion.
(iv) Shear stress in foundation.

For an embankment to be stable, there should be friction and cohesion between the particles. (Friction is great for coarse soil, and small for fine soils). A small addition of moisture increases friction. The cohesion gives additional stability to earthwork. It is a varying force, which generally increases with a moderate amount of moisture. Hence for an embankment to be stable, there should

be limited quantity of moisture, that is, all extra moisture should be drained off.

Stability also depends upon the nature of the formation and stratification of the embankment. A consolidated bund consisting of layers is suitable and satisfactory. For a dam to be stable, it must resist the water pressure, and also the foundation must be able to stand the weight of the dam. The stability also depends upon the shear strength of the particles of soil, and this in turn, is either due to cohesion or the friction of the particles, or both. If a structure is to be stable, it must be watertight and non-slipping.

If the consolidation or the compaction of layers of the embankment of earthwork is to be satisfactory, the soil should contain proper proportions of clay, sand and silt in it. This proportion will make the soil cohesive and the embankment will then be stable.

The lower portions of an earthen dam are subjected to different conditions from the upper portion. They are highly compressed and also contain more moisture. Due to extra moisture the cohesion is also less here and so also friction. Hence high embankments will have to be treated separately from low embankments, that is, they will have to be made more stable; extra moisture should be removed from them, to increase the friction and cohesion, and for this purpose, stone pitching to the rear slope and clayey soil to the front slope (to make it water-tight) are resorted to.

7. Percolation: Percolation is the movement of ground water which is brought about by the force of gravity. It is the gradual flow between and through the particles of soil.

Percolation through different soils:

(i) *Rock:* All rocks, unless they are decayed or disintegrated, are impermeable, only if there are fissures or cavities, percolation takes place through them.

(ii) *Gritty soils:* Gritty soils are soils like decomposed rock, shingle, gravel. These are freely permeable.

(iii) *Plastic soils:* These are slightly permeable and damp. They cannot be absolutely impermeable.

8. D'arcy's Law: D'arcy's formula is

$$Q = \frac{KAH}{L} \text{ and } Q = AV$$

$$\text{Therefore, } AV = \frac{KAH}{L} \text{ or } V = \frac{KH}{L}$$

This law is applicable only for slow motion where Reynold's number is less than one. Reynold's number is given by

$$R = \frac{dv\delta}{\mu} \text{ in which}$$

d is the average diameter of the grains,
v is the average velocity of flow,
δ is the density of water
μ is the absolute viscosity of water and
R is Reynold's number

For the proper calculations of the underground flow of water, the analytical method,—using Laplace Equations, etc.,—is very cumbersome and tedious; so some other methods are employed. They are (1) Graphical Method
(2) Hydraulic Model Method
(3) Membrane Analogy Method
(4) Electric Analogy Method.

Electric Analogy Method: The electric analogy method, as the name itself indicates, is solely based on the analogy between the flow of electric current and the flow of water as is seen from the D'arcy's Formula and Ohm's Law.

The D'arcy's Formula for the flow of water through soil is

$$Q = \frac{K.\ A.\ H.}{L}$$

where, Q is the discharge in a given unit of time.
K is the coefficient of permeability for the given material.
A is the area of the soil mass through which the flow takes place.
H is the difference in head
and L is the length of the path of percolation.

The Ohm's law for the flow of electric current is

$$I = \frac{k.\ a.\ V}{L}$$

where, I is the current flowing in a given unit of time.
k is the coefficient of electric conductivity for the material.
a is the cross sectional area of the conductor
V is the difference in Voltage
and L is the length of the path of current.

As shown above, with the close analogy between the flow of water and the flow of electric current, electrical Models are used to determine the underground flow of water.

Pavlovsky a Russian, was the first to attack the problem of flow of water through subsoils (of hydraulic structures) from the analogy of flow of electricity through a conductor. He did this work mostly as an academical one, and there was not much use made by engineers, as the results did not agree well with the actual field work.

The electric analogy method works on the principle of the Wheatstone's Bridge. The Electric model used for this purpose consists of a glass sheet, on which the section of the dam under consideration is drawn in pencil. Strips of pyralin are cemented to the glass vertically with acetone along the boundary lines to form a tray, or a glass tray itself can be used. The phreatic or the saturation line is roughly made with modelling clay, and the tray is filled with salt water or tap water to act as an electrolyte. A thick copper, or brass strip electrode placed along the upstream slope ensures a constant potential, while small strips of brass or copper strips connected in series with resistors along the downstream slope, give a variable potential.

The whole arrangement is connected in parallel with the main resistance of an electric circuit with the connections as shown in Fig. 72. In the cross connection (as shown in fig.) a cathode ray null indicator consisting of a variable central trap connected to the resistence through the cathode ray tube is connected to the wire joining the probing needle to the tapped resistance.

Before drawing the flow net, the final position of the saturation or the phreatic line has to be fixed. For this purpose, the potential indicated by the voltmeter should be proportionate to the difference in height, since the drop in head at any point along the saturation or phreatic line should be equal to its depth below the full tank level. Hence the phreatic line has to be shifted until the required

difference in potential is indicated at that point, if it was not so in the previous case. In the same way more points on the phreatic line are tested and fixed, thus fixing the whole line. Then the

ELECTRIC ANALOGY METHOD TEST

ABCD is the bund

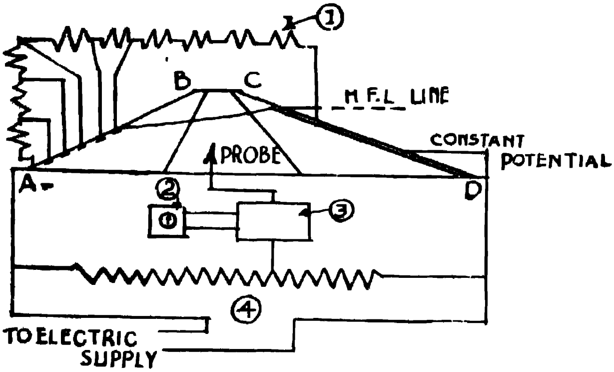


Fig. 72.

(1) Resistors for giving Varying Resistance (2) Cathode Ray Null Indicator or Galvanometer (3) Voltage Amplifier (4) Tapped Resistance.

points having constant potentials as read by the probing needle, which is shifted from point to point, are located, thus tracing equipotential lines.

Then the potential drop is altered by altering the number of resistors on the downstream slope which are put in circuit according to the requirements (by the junction keys). Equipotential lines are drawn for each change in the drop of potential, by tracing with the probing needle, thus to get the flow net.

Note :—Instead of the cathode ray null indicator, specially devised earphones can be used to adjust the proportionate voltage drop, which is obtained when the adjustment of the resistance silences the earphone. Recently Dr. Vaidyanathan has shown that the distribution of pressure under works on sand foundation can be exactly reproduced on hydraulic or electric models.

D'arcy's formula does not take into consideration the viscosity of water which varies with the temperature. Therefore

Hazen's formula is developed over it. This is given by

$$V = 3{\cdot}28\, Cd^2 \frac{h}{l} \left(\frac{t+10}{60}\right)$$

where V is the velocity of flow in ft. per day of a column of water of the same area as that of sand,

d is the 'effective size' of the sand grains,

C is a coefficient, which is usually taken as 1,000

$\frac{h}{l}$ is the hydraulic gradient

$= \frac{\text{head producing the flow}}{\text{length of the coloumn of sand}}$

t is the temperature of the water in degrees F.

This formula is applicable where the effective size is between 0·10 and 3·0 millimetres.

In Southern India, t is taken as 80° F, and then

$$V = 315 d^2 \frac{h}{l}.$$

The stability of a work is not affected by percolation through or under it, except under the following conditions :—

(a) The percolation is of sufficient velocity to carry off with it particles of soil.

(b) Pressure developed by and due to percolation is able to fracture and displace any portion of the work.

The percolation velocity is proportionately greater in a coarse stuff than in a fine one, because the velocity required to displace the coarse material is greater. For the velocity of percolation to be within a safe limit, the formula used is $S = h/l$, where, S is the hydraulic gradient, h is the head and l is the minimum length of percolation. For sand, it varies from $\frac{1}{15}$ to $\frac{1}{12}$.

The flow of percolation is only by gravity. This flow varies with the permeability and the velocity.

N. B.—The percolation velocity is very small and is expressed in feet per day. For example, 4 ft. per day, for 10 ft. head for sand.

If the foundation of a dam is permeable, water percolates under the dam. This percolation is not due to gravity, but is due to the pressure of the head of water stored in the reservoir. It will

be steeper, as the head is greater, and may prove unsafe in cases and to flatten the percolation gradient, the base width of the dam is made sufficiently wide, or in the alternative, a *cut-off wall* is constructed at the centre on a hard bed. This makes the percolation path longer (Vide Fig. 73) and thus the gradient flatter, and as a consequence the velocity is reduced.

CUT-OFF WALL IN THE CENTRE OF AN EARTHEN DAM

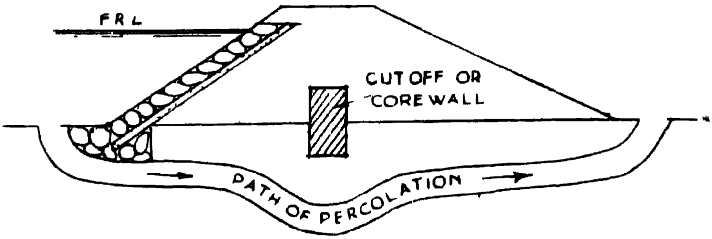


Fig. 73.

Sometimes in place of the cut-off wall, a *blanket* of watertight material is provided on the upstream side of the bed inside the dam. (See Fig. 74).

EARTHEN DAM SHOWING BLANKET

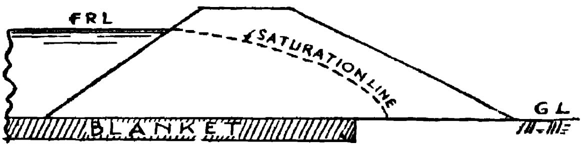


Fig. 74.

Purpose of Blanket is

(i) to increase the length of the path of percolation for seepage under the dam, and thus

(ii) to decrease the velocity and quantity of seepage.

The thickness of blanket is generally from 5 to 10 ft.

The length of the path of percolation under the blanket + impervious portion of dam.	8 or 10 times gross head.

The length of the impervious blanket in a tank bed is given by the formula

$$x = \frac{khd - pqb}{pq}$$

where, x = length of impervious upstream blanket in feet,

k = mean horizontal permeability coefficient of the pervious underground.

h = gross head in feet on impervious upstream blanket.

d = depth of pervious underground, in feet,

p = percentage (stated as a decimal) of flow under the dam without a blanket, to which it is desired to reduce the seepage by means of the blanket,

b = length of impervious portion of length of dam,

q = flow under dam without a blanket per foot of dam = k (h/b) d. For problems – Vide Book of Authors 'Solutions of Problems in Irrigation Engineering.'

9. Saturation Gradient: It is the inclined line which starts from a point, where the H. F. L. cuts the front face of the dam and passes through the top portion of saturated plane of the dam. If the material of the dam is watertight, this line will be steep: and this becomes flatter, as the watertightness becomes less and less.

Every soil has its own saturation gradient. If it is compact, this gradient is steep; and if it is a masonry wall or even a puddle wall, the saturation gradient is much steeper than in other earthwork section; hence the provision of a core wall in an earthen dam. Also a filter on the rear toe affects the saturation line. See Figs. 75, 76 and 77.

ORDINARY HOMOGENEOUS MATERIAL USED

Fig. 75.

CORE WALL USED

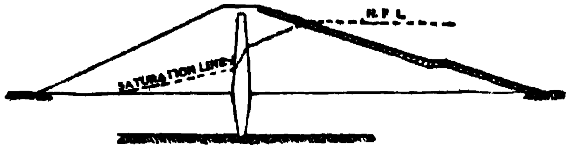


Fig. 76.

THE FLOW LINE AT ENTRY IS AT RIGHT ANGLES TO THE SLOPE AND AT THE EXIT AT RIGHT ANGLES TO THE BED

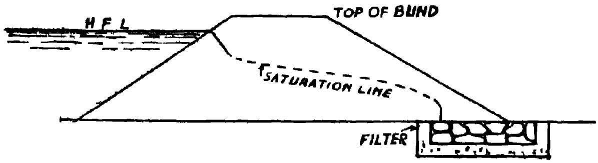


Fig. 77.

The foundation is impervious and the embankment is made up of homogeneous material and there is drainage or filter provided at the rear of the base.

Stability of Earthen Dams (Testing)

For the stability of an earthen dam, stability for the entire dam as a whole and for the two slopes the front and rear, should be taken into consideration. The internal stability of the dam is ascertained in various ways. The most common method among them is the *slip circle method.*

Slip circle method—When a slip occurs in an embankment, the surface of the cavity formed is not a plane but it is curved. The actual profile is not a circle; it is more correctly a cycloid, or a logarithmic spiral, but yet it is taken as a circle for all calculation purposes. The slip circle is the most dangerous circle along which the earth may slip. The stability is tested along the various circular arcs and that which gives the least resistance to shear is taken into account.

The factor of safety is taken between 1·3 and 1·5 and this is found to be safe, as in course of time earthen dams consolidate

well, and cosequently the factor of safety may rise to even 2.

$$\text{Factor of safety} = \frac{\text{Shear strength}}{\text{Shear stress of circle (having the least stability)}}.$$

If the factor of safety is high, it shows that the slope could be made steeper and economy effected. If impervious fill is used, the angle of internal friction is small and as a result, factor of safety is decreased.

In order to find the factor of safety against slipping along the slip circle, we have to consider the various forces acting on the wedge formed by the slip circle.

The main process consists of considering a very small or infinitely thin slice in the wedge formed by the slip circle and finding the algebraic sum of all the tangential and normal components of the forces acting on it, and from these values arriving at a factor of safety for the whole wedge (which comprises a number of such thin slices.)

Let an arc PQ (See Fig. 78) of a circle represent the slip

SLIP CIRCLE

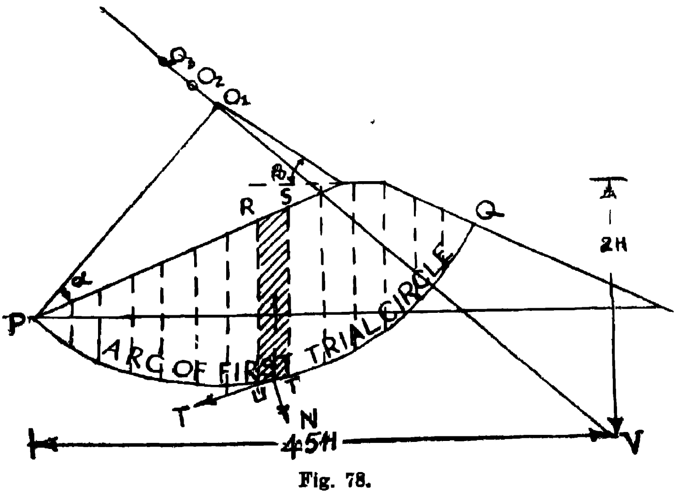


Fig. 78.

circle, the material in the wedge PRSQTU will slide, when a slip occurs. The wedge can be divided into a number of thin slices and let PQRS be one such slice. The forces acting on this slice are

(1) Weight of the material, and

(2) Pore pressure; this is the hydrostatic pressure of water acting in all directions in the pores of the material, below the seepage line and it is proportional to the depth of the point below the seepage line. The effect of this force is to cause uplift or buoyancy along the arc of the slip circle, thus reducing the normal component of the weight and not affecting the tangential component. The pore pressure can be found out by drawing the flow net for the embankment. But in actual practice, it is taken as a certain multiple (1 to 1·5 for the U/S and 0·5 for the D/S) of the impervious fill, depending upon (a) the nature of the fill which affects the permeability and (b) on the rapidity of the drawdown.

(3) The Frictional resistance depending upon the internal angle of friction of the material and the net normal component and the cohesive strength reckoned over the whole length of the arc, comprises the shear strength. It is sufficient, if we consider the forces alone and not their moments about the centre of the slip circle, since all these forces have the same moment arm except in cases where there is external loading (such as a building, etc.,).

As the analytical calculation is difficult, the normal and tangential components of the weight of the wedge are found by the application of calculus, and the corresponding Pore pressure along these directions is determined graphically.

If N and T are the resultant normal and tangential componenents on a thin slice under consideration, then the factor of safety will be $\frac{\Sigma N \tan\phi + cL}{\Sigma T}$ where cL is the cohesive resistance along the arc of the slip circle.

If we take the water pressures on the upstream and downstream faces (if any) also into account, then the factor of safety will be given by $\frac{\Sigma [N_w - P + N_p] \tan\phi + cL}{\Sigma [T_w + T_p]}$

where N_w is the normal component of the weight of the material in the slice (in the wedge.)

P is the Pore Pressure

N_p is the normal component of the water pressure on the face of the dam.

ϕ is the internal angle of friction of the material

c is the cohesion in the soil

L is the length of the arc of the slip circle.

T_w is the tangential component of the weight of the slice

T_p is the tangential component of the water pressure on the face of the dam.

Location of the slip circle: The slip circle which gives the worst and the most dangerous result must be considered in order to find the factor of safety against slipping. The centre of the critical circle can be found approximately in two ways as indicated by Fellinius.

First Method: As in fig. 78 the angles α and β which can be obtained from tables have to be set to get the centre 0_1 of the first trial circle. For the subsequent centres, a point V at 4·5 H distant from the toe of the embankment is located at a depth "H" below the top of the bank, (where H is the height of the bank,) and on the line joining this to 0_1; other trial centres are also taken and tried.

Second Method: The centre of the first trial circle is located by finding a point at which the angles subtended by the slope of the bank intercepted by the arc of the circle is $133\frac{1}{2}$ degrees. To locate the subsequent centres with this point 0_1 as centre, circles of radii H/8, H/4, 3H/8, H/2 etc., are drawn and radial lines at 30°, 60°, 90° and 120° are drawn, and the intersection of these with the circles, gives the centres of the trial circles. (See Fig. 79).

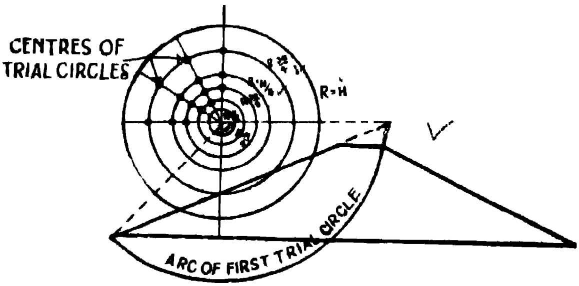


Fig. 79.

The values of the factor of safety as obtained from the various trial circles are tabulated in order to have a comparative study, so as to review the whole at a perspective glance.

This method is only a guide in checking the stability of the earthen dams and cannot be taken as absolutely correct on account of (1) the assumptions made in arriving at it, which are only rough (these assumptions are average values of both the shear stress and strength which are taken into account; it comes to mean, that failure may occur all at once) and (2) The pressure on the slip circle is not affected by increasing the height of the dam.

Stability and soils used for dams: If the soil used for the dam is non–cohesive, i. e. sand, the stability depends upon the angle of internal friction. This angle varies from 32° to 40° and it depends upon (a) angularity of the particles and (b) density of packing; here stability is independent of the height of the dam.

If the soil used is cohesive, stability depends upon both the angle of friction and the height of the dam.

Hence calculation of the stability of the dam should be done (a) by the slip circle method, for important works, and (b) by using Taylor's stability numbers, for small works.

The following points should be kept in view in the design of an earthen embankment:—

10. Design: Until quite recently, the design of earthen dams has been more of an art than a science, but tanks to the development of soil mechanics the design and the construction of earthen dams are done with greater assurance and economy.

(i) There should be no danger of the overtopping of the bund: for this purpose, sufficient spill-way capacity and sufficient free board should be provided.

(ii) The seepage line should be well within the downstream face.

(iii) The upstream face slope should be safe against draw-down.

(iv) The upstream and downstream slopes should be flat enough, so that with the materials utilised in the embankment, they will be stable and show a satisfactory factor of safety by recognised methods of analysis.

(v) The upstream and downstream slopes of the earthen dams should be flat enough so that the shear stress produced in

the foundation is less than the shear strength of the material in the foundation to arrive at a suitable factor of safety.

(vi) There should be no opportunity for the free passage of water from the upstream to the downstream face.

(vii) Water which passes through and under the dam, when it reaches the discharge surface, should have a pressure and velocity so small, that it is incapable of moving the material of which the dam or its foundation is composed.

(viii) The upstream face should be properly protected against wave action and the downstream face against the action of rain, and wear and tear by men and animals.

An Earthen Dam consists of (i) the dam or bund proper, (ii) and outlets or sluices, and (iii) the weir or surplus work.

N. B.—The dam proper will be dealt with in this section, and the outlets and weirs in separate chapters.

11. The Dam: *The section of the dam* depends upon, (a) the top width, (b) the slopes and (c) the height. When these three are fixed, the section is automatically fixed. In any particular Dam, it is an advantage to preserve uniformity of section throughout its length. Sometimes for economy's sake, attempts are made to reduce the section of the dam in some places. This is not advisable, as it not only spoils the general appearance of the dam, but it is also not very economical.

If any reduction in section is to be made at all, the three convenient positions for it in the length of the dam are

(i) The gorge embankment portion,
(ii) The high portion of the embankment and
(iii) The low flank of the embankment.

(a) *Top width:* The top width of the dam varies with the size of the reservoir and with the height of the dam. Sometimes to allow of the top level of the dam to be restored to its original level, should any extra settlement occur, the width is made more than that required. Again, if it is intended to allow any traffic on the top of the dam, it is made wide enough for the purpose. It is always a good policy to maintain the top width uniform throughout; but generally, this top width is not made uniform, but is reduced in width at the flanks.

Usually the top width of the bund ranges from 6 to 10 ft. For large tanks they are made specially wider.

A thumb rule sometimes adopted is that the top width b is equal to $\frac{h}{5} + 5$ where h is the height of embankment, and another rule is, the top width is equal to $\frac{1}{4}$ the depth of water at the place.

(b) *Slopes*: The slopes of the dam depend upon the nature of the material to be used for the dam. The side slopes usually adopted are as guided by experience, and they are sufficiently flat and could be relied upon to resist slipping. The upstream slope is made flatter, as it consists of more clayey material and is saturated by the water in the tank; and the outer slope is made just flatter than the angle of repose.

In practice, however, the front slope is not left natural, but is provided mostly with rough stone work either revetment or pitching, and as a consequence, the slope is made steeper and the section reduced (for economy).

(c) *Height of the dam*: (i) The height of a dam depends upon the ground level at the place.

(ii) It also depends upon the H. F. L. of the reservoir, as the top of the bund is to be kept beyond the reach of the H. F. L.

(iii) It also depends on the amount of free board, provided for the dam. This free board varies with the importance of the dam, say from 4 to 10 ft. This is necessary, as a matter of safety in ordinary circumstances, and as a provision for any accidental settlement of the bund which may occur.

(iv) It also depends upon the amount of foundation clearance, because the deeper the foundation is taken below the ground level, the higher the dam becomes.

(v) Provision should also be made for the action of waves, in deciding the height of the dam. The directions of the prevailing wind should be studied, and when the wind blows directly towards the bund, the top level should be kept specially high, so that it may be beyond the reach of the highest wave. This should be done with care, especially where the bund is deep, as the damage here, if any, will be enormous.

Breaching Sections: For important dams, what are called "*breaching sections*" are provided. These are introduced to guard

against the top of the dam over-flowing and breaching at awkward places. These breaching sections are made smaller in cross-section. That is, the top width is reduced, the top level is also kept purposely lower by 1 ft. or so, so that the flood water may over-top here, and the side slopes are also kept steep (to be safer under ordinary conditions). They are located where there are natural saddles on the ridge line, and if a saddle does not exist, a channel is artificially formed, to lead off the flood safely, away from the dam. In the case where the waste weir is situated in an independent valley or saddle, the flank embankments are made to serve as the breaching sections. It is not, however, necessary that breaching sections should breach automatically, because the flood always takes time to rise, and there is good warning. It is enough if these breaching sections are so reduced in area, as to offer facility for being cut away rapidly when required.

Allowance for settlement: No matter how well a dam has been consolidated during construction, its enormous weight, which far exceeds any that can be artificially brought to bear on it, tends still further to compress it. This settlement also occurs from moisture and rainfall. Hence provision should always be made for the subsequent settlement, when setting out for the work.

The final settlement will be practically attained in a few months after the full depth of storage comes against the dam, and it will continue to settle very slowly for a few years more, (say 5 years) and after this period, there is not generally any appreciable further settlement at all.

N. B.—The vertical settlement of a well-consolidated dam is usually taken as 1/30 of its total height measured from the clear foundation to the designed top.

12. Classification of Earthen Dams: Earthen dams may be classified as: (i) pure earthen dams, (ii) compound dams, (iii) composite dams, and (iv) American types of dams.

(i) *Pure Earthen Dams*: As the word indicates, these are dams, thrown across streams and are constructed with mere soil only, which is available at the site (Of course proper selection should be made of the different soils to make the dam watertight). The slopes are made greater than the angle of repose, and as flat

as practicable, consistent with economy, to be stable against water pressure.

(ii) *Compound Dams:* These are pure earthen dams, as said above, but in addition, they are provided with dry stone revetment or pitching, mostly at the front or water surface; so that, the front slope may be made steeper and some earthwork thereby saved. The rear slope is, as usual, provided with turfing.

The advantages claimed by compound dams are

(a) They protect the slopes from the effects of water and rain.

(b) They can be constructed to great height,

(c) They can be raised by a good amount, when required,

(d) This type is more expensive than the pure earthen dam, but is cheaper than a masonry dam.

The advantage it has over a composite dam, is that its hearting is continuous and uniform throughout, so that there is no tendency for the formation of leakage planes through it.

(iii) *Composite Dams*: These are dams in which part of their length is earthen embankment and part masonry dam.

They are best suited for sites, where the crossings are deep and the flanks are on high ridges.

In respect of cost, these are midway between earthen dams and masonry dams.

N. B.—The junction between the earthwork and masonry should be very carefully made.

(iv) *American type of Dams*: These may be divided as follows :—

(a) Rock-fill dams, (b) Hydraulic-fill dams, (c) Earthen embankment with steel plate core walls, (d) Earthen embankment with masonry core walls.

The first two, that is, the Rock-fill dams and the Hydraulic-fill dams are dealt with separately.

(c) *Earthen embankment with steel plate core walls:* When steel plate core walls are used, they are sometimes protected by casings of asphalt concrete construction, but these have been known to slip off during expansion. These are not used except in special cases and are of use only in America.

(d) *Earthen embankment with masonry core walls*:—A good example of this type of dam is the Croton Dam of New York. (See Fig. 80).

EARTHEN EMBANKMENT WITH MASONRY CORE WALL
(CROTON DAM)

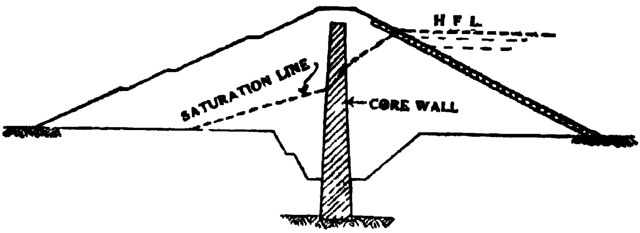


Fig. 80.

The advantages of this type are:

(i) If founded on an unyielding and impervious stratum, the masonry wall effectually cuts off the leakages, in the centre of the dam, preventing percolation to the downstream face.

(ii) When leaks occur, the masonry stands better than a puddle core wall.

(iii) It forms a suitable junction for the two casings of the dam, which are usually constructed of different kinds of materials. Without it, cracks may occur at junctions, owing to unequal settlement.

(iv) It makes the outlet culvert to be carried through the dam, with perfect safety.

(v) It enables the outlet tower to be dispensed with, and to be replaced by a dry wall built upstream, or in connection with the core wall, thus dispensing with the head of the outlet tower.

(vi) It gives an earthen dam greater strength to resist the erosion action of water passing over its top. (This however should never be allowed to happen, if waste weir provisions are ample).

13. Types of Earthen Dams: The types of earthen dams generally adopted are given below:—

(i) The whole section of the bund is formed of uniform and

homogeneous material which is retentive but not permeable, See Fig. 81.

EARTHEN DAM TYPE 1.
ONE HOMOGENEOUS MATERIAL

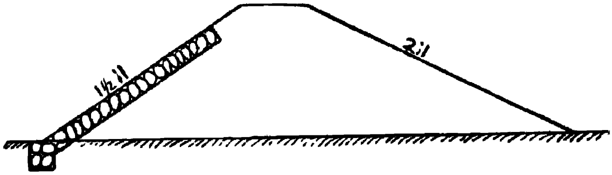


Fig. 81.

This type is generally used throughout South India, but the materials used are not always best suited for the purpose or are not selected properly, (perhaps due to economy), and consequently a number of failures or breaches occur during the heavy rains.

(ii) The second type used is, hearting of a plastic material at the centre with a casing of more stable material with a large quantity of grit on the two sides, both inner and outer.

This type has been generally in use, even for large dams in the Bombay State. (It is now coming into practice also in Madras).

This type may be advantageously used in sites where the top soil is black cotton for a depth of 3 to 4 ft. overlying decomposed rock; that is, the soil is mainly grit, but with a small quantity of plastic material. If the section is fully of plastic or clayey soil, it will crack when it becomes dry, and when it is full of gritty soil, there will be free percolation through it ; but when the section is composed of plastic soil for the hearting and gritty soil for the casing, as in this type, the plastic soil will stop percolation and the gritty soil will protect the hearting from cracking. Thus the materials are disposed in positions which are actually required, (See Fig. 82).

EARTHEN DAM TYPE 2.
HEARTING OF PLASTIC MATERIAL AND THE TWO CASINGS OF GRIT

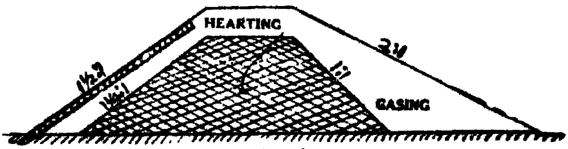


Fig. 82.

(iii) In this type, the section is formed with an impervious core wall at the centre. This core wall may be either of puddle clay where the depth is small, or of masonry where the depth is great, (say more than 40 to 50 ft.); the outer casings are of ordinary earth. In some cases, the front or the water face is protected with rough-stone work. (See Fig. 83).

EARTHEN DAM TYPE

IMPERVIOUS CORE WALL AT THE CENTRE

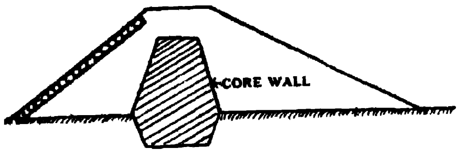


Fig. 83.

14. Construction of an Earthen Dam : The dam will generally consist of an earthen embankment with puddle or concrete core wall at the centre of cross-section arranged according to a selected design.

The following preparation should be made before the commencement of the dam work.

(i) The land taken up for the work should be properly demarcated (and a plan and register kept on the work).

(ii) The soil and other materials available for the embankment, puddle trench, etc., and the sufficiency of each description of the material required (as well as the cost of carriage) should be considered, and the best selection consistent with economy determined.

Detailed drawings of the puddle trench and the working section of the dam, with the widths marked at different heights, should be maintained.

Clearing site and preparing foundation :—The whole site must be cleared, to the full width of the dam, of all trees, shrubs, grass, rubbish of all sorts, loose stones and soft stuff. All roots should be dug out and the loose surface-soil scraped off to a depth of 6 in. minimum or till firm natural soil is obtained. If the natural soil in any part of the seat of the embankment is found to be compressible, or to contain salt or otherwise have a suspicious

appearance, the same should be removed before commencing the work. All decayed and undermined strata should be removed and

FOUNDATION FOR EXCAVATION IN ORDINARY SOIL

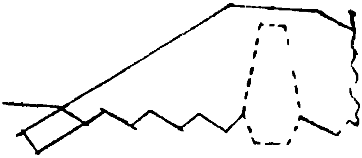


Fig. 84.

necessary grips formed in the bed, to receive the new soil for embankment. See Fig. 84. When soft rock is met with the foundation should be excavated as shown in Fig. 85.

FOUNDATION FOR EMBANKMENT
IN SOFT ROCK

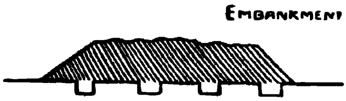


Fig 85.

Disposal of the excavated materials: Rock and debris excavated from the concrete and puddle trenches may be used again if found suitable for the construction of the outer slope of the dam. If they are not fit for use here, they shall be removed away from the site, so that they will not interfere with the work, and also that they may not be used for the work proper.

Construction: At the centre of the bund, to prevent percolation, a core wall of puddle or masonry or concrete is constructed, (details of construction of core walls are dealt with separately).

Provision should be made for drawing the water leaking beyond puddle trench. This is to prevent the saturation of the rear toe of the dam or the ground underneath, or immediately beyond the rear toe. The base of the dam to the rear of the puddle trench, should, as far as possible, be kept dry and firm; if not, slips and settlements will occur in the rear slope. The amount of leakage varies with the nature of the subsoil and with the depth and pressure of water against the dam. This pressure is the greatest

in the river bed, and hence special attention is to be paid to this portion.

All the leakage shuld be collected and led away, clear from the rear toe, by drains constructed at the ground surface below the dam, or by filling for at least 2 ft. the whole base of the dam in the rear of the puddle trench with large rubble stones. (See Fig. 86).

GENERAL CROSS-SECTION OF AN EARTHEN DAM

SHOWING PUDDLE AND DRAINAGE

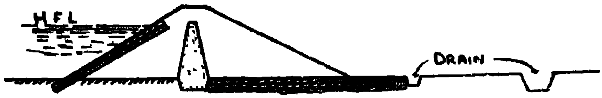


Fig 86.

All pools close to the outer slope of the dam, should be thoroughly drained, (see also drainage under dams).

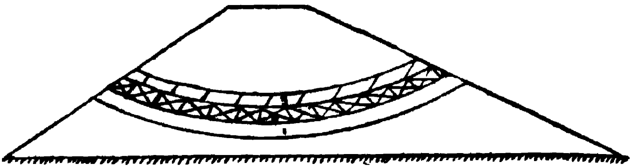
15. Embankment: (a) *Construction:* The embankment is to consist of watertight hearting of clay, or clayey earth with casing on both sides. Clay hearting is intended to make the dam watertight, and the casing is to afford protection to the hearting, against the effects of alternate saturation and extreme dryness.

(a) *Materials:* The materials for the hearting should be good clay or retentive earth. A good mixture is $\frac{1}{4}$ gravel and $\frac{3}{4}$ local earth, or one part of pure black cotton soil or other clay to one part of pure gravel. This should be available as near to the locality as possible, so that the work may be done economically. The hearting material should be of the same quality throughout, so that, there may not be unequal settlement. This material should be free from salts, large stones, and rubbish of any sort. The casing should be made of clay and gravel, preference being given to a natural mixture of good earth.

(c) *Forming embankment and consolidation:* The material used should be first freed of all clods and lumps. It should be laid in continuous layers with slight slope towards the centre, making a concave curve, the centre being 3 to 6 inches lower than the outer edges (See Fig. 87). The material should be spread in even layers of 5 or 6 in. in thickness, so that when rolled it will settle to about 4 in.; but when a steam roller is used, this thickness may be incre-

ased to about 9 in. All the earth used in the embankment should be such, as is naturally moist in itself or made so, before it is spread.

CONSTRUCTION OF EMBANKMENT, FORMATION OF LAYERS



1, 2 and 3 Layers of Embankment.
Fig. 87

No watering should be allowed until the layer has been completely rolled. Water should be sprinkled to sink into the completed layer, before the next layer is spread.

Rollers may be of stone or of iron and of such a size and weight that they will give a pressure of $\frac{3}{4}$ to $\frac{1}{2}$ a ton per foot length of roller. It is sometimes advisible to pass a light roller, over a newly spread layer, in order to bring it to a level surface, before working a heavy roller. To prevent the materials sticking to the rollers, dry earth should be sprinkled on the surface, before or during consolidation. The roller should be kept clean using sprayers.

In places where rollers cannot be used, the consolidation should be done with heavy tampers or ponners, worked by lines of men moving backwards and forwards on the surface till the layers are thoroughly consolidated. It should be noted that when ramming is done by manual labour, the layer should not exceed 3 in. in thickness. No matter however well consolidated a dam may be, it will settle after filling. Therefore an allowance of 1/20 to 1/30 more of its total height from the foundation should be allowed in all works.

16. Slopes: *Outer slope*: When it is proposed to protect the outer slope of an embankment with *hariali* or other creeping grass, the outer casing is to be finished off, with a top layer containing a sufficient admixture of soil (say sandy soil) favourable for the growth of grass.

Inner slope: The inner slope of an embankment shall be protected with stone pitching or revetment. When circumstances

permit, one rainy season should be allowed to elapse, after constructing the slope, to allow for consolidation before the stone work is commenced.

17. Final Closure of the Dam: When a new dam is being constructed across a stream, provision should be made for the flood discharge during the progress of the work. That is, a flood gap should be left in a convenient place and this gap should be filled up finally.

There are two ways of attacking this problem, according to the magnitude of the work.

(i) Work small and (ii) Work big.

If the work is of small magnitude, the flood gap is allowed in the centre or the river portion of the dam, and this is filled up in one season itself (See Fig. 88).

FILLING THE GAP IN EARTHEN DAM

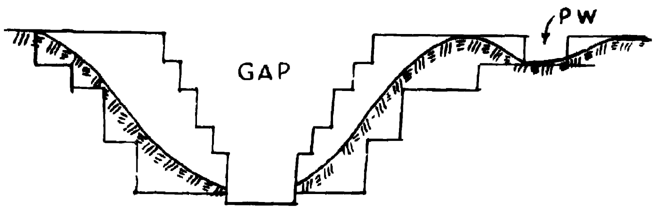


Fig. 88.

If the work is of great magnitude, and if the flood gap is left in the river portion itself and cannot be filled up in one season, the following method is adopted.

The centre of the dam in the river portion is attacked for half the width, the other half being allowed to pass the ordinary discharge. The gap is attacked only during the driest season, so that the discharge to be tackled will be the minimum. While attacking the other half, a temporary gap or cut is made at one flank and this portion is filled up. The next season—dry season—some more height in the river gap is raised and another temporary gap is formed for the discharge of flood water. Thus flood gaps are left every year, and, in the succeeding year, the lower flood gap is closed and a higher one created, as the central portion is raised, and

finally the permanent waste weir is constructed and the remaining portion of the central gap filled up and completed. (See Fig. 89).

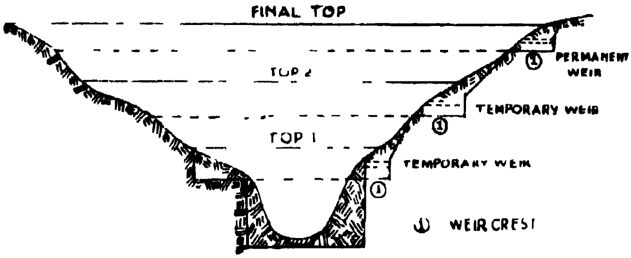


Fig. 89.

The advantages of this method are (i) the central portion which is deep is not raised in one season and the work is done slowly, (ii) the settlement is gradual as the water is stored in the tank, (iii) the storage of water in the bed is very advantageous, not only for the work proper, but also as water-supply to the people employed on the work.

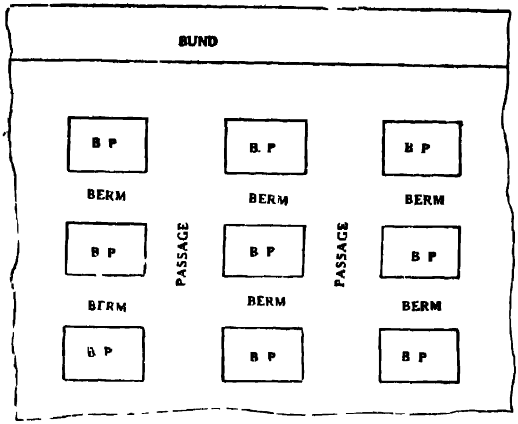
18. Borrow Pits: Borrow pits should not be excavated at random. They should be done, in accordance with some previous understanding, getting a plan prepared for the excavation of these pits. The quantity of earthwork required in each length of the dam is known from the estimate, and it is economical to get this quantity, in the bed of the tank opposite to this length only. This will save the conveyance charges to a good extent.

N. B.—As the quantity of earthwork is given either in c. yds. or in 100 c. ft., it will be easy for calculation, if the length multiplied by the breadth of any pit is a multiple of 27 or 100.

All the pits required for the selection of the bund should be regularly excavated parallel to the bund, leaving a margin, say of 6 to 9 ft. between them for workmen to pass. Again between sections, it is a good practice to allow a margin or berm, (of say, about 10 to 12 ft.) for facility of work, that is, for conveying the material. The berm for passage will be at right angles to the line of bund. (See Fig. 90).

No pit should be excavated very near the toe, either towards the front or rear of the bund. An understanding is, that no pit in the bed of the tank should be excavated nearer than thrice the

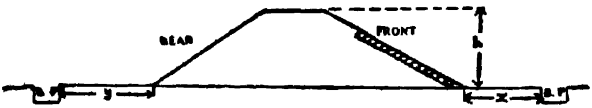
SKETCH SHOWING LOCATION OF BORROW PITS



B. P.—Borrow Pit

Fig. 90.

height of the dam from the rear toe and nearer than twice the height of the dam from the front toe to the bund (See Fig. 91).



$x \nless 2h, \ y \nless 3h.$

Fig. 91.

Also no pit should be excavated more than 5 ft. deep from the front toe, within a distance of 100 yards. No porous strata should be exposed or uncovered.

19. Revetment: Revetment is dry stone work constructed to an embankment, to protect it from wear and tear, due to action of weather or due to the use of the slopes by men and cattle.

Revetment is generally constructed with a slope of $1\frac{1}{2}$ to 1 or 2 to 1 for ordinary soil, and $2\frac{1}{2}$ to 1 and 3 to 1 in special soils. It may be stepped or it may be constructed of different slopes, vertical at the top and sloping at the bottom.

The slope of the bank should be trimmed to the proper slope, before the revetment is constructed. The stones for revetment should be laid closely in position and firmly bedded, and they

should be perpendicular to the slope. Packing of revetment is important, but is generally neglected or not much care is bestowed on this item, and so this work should be properly supervised. This packing should be of small stones closely packed and should be carried up simultaneously with the main revetment work. The toe of the revetment should be secured and prevented from slipping as shown in Fig. 92.

REVETMENT OF AN EARTHEN DAM

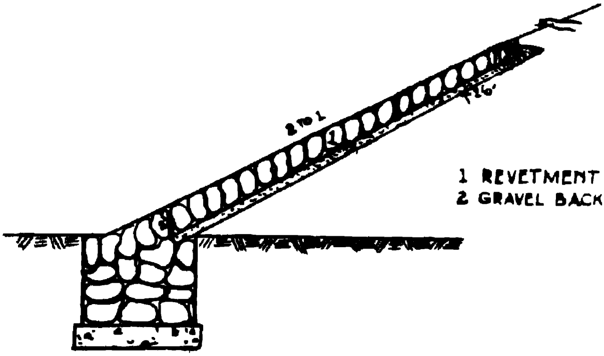


Fig. 92.

The stones to be of fairly good sized, hard, quarry or boulder stone, such as will not weather on the surface. They are to be roughly hewn or squared with the hammer, to ensure the stones fitting fairly, one to the other, so as not to expose the earthwork below. The stones are to be laid with their broadest faces downwards with their length into the work and at right angles to the slope and firmly bedded on a layer of gravel varying from 9" to 12" in thickness. The stones are to be packed against each other with a hammer or mallet, so as to fit closely for at least 3 in. in height and to lie generally perpendicular to the slope. No pinning is to be allowed between the sides of stones, and the use of chips should be confined to hollows only, and inequalities in the bed and for packing after the stones are laid in the surface, to form a uniform slope.

The slope is to be now and then tested, with the help of a sloping board.

Continuous joints should be avoided, especially when Voddars are employed and gangs are given different *task* or different lengths of the bund.

Enough chips for jelly-backing and filling the interstices, should be got collected beforehand, otherwise, the work is always scamped, with the result, that a lot of hollows will be left and the revetment will sink in these places.

For revetment work used for earthen dams across storage reservoirs, the thickness of revetment is made to increase with the depth, the minimum thickness at the top being 1½ ft. at right angles to the slope, and increasing by 3 in. in thickness, for every vertical depth. Vide Table below.

TABLES SHOWING THICKNESS OF REVETMENT FOR DIFFERENT HEIGHTS

Vertical height	Slanting height	Thickness of revtt.
0	0	1·50
1	1·8	1·58
2	3·6	1·65
3	5·4	1·72
4	7·2	1·80
5	9·0	1·88
6	10·8	1·95
7	12·6	2·02
8	14·4	2·10
9	16·2	2·18
10	18·0	2·25
11	19·8	2·32
12	21·6	2·40
13	23·4	2·48

Slanting height	Thickness of revett.
0	1·50
1	1·54
2	1·58
3	1·63
4	1·67
5	1·71
6	1·75
7	1·79
8	1·83
9	1·87
10	1·92
11	1·96
12	2·00
13	2·04
14	2·08
15	2·12
16	2·16
17	2·20
18	2·25
19	2·29
20	2·33
21	2·37
22	2·42
23	2·46

20. Pitching: Pitching is a covering of hard material for a soft interior soil, to protect it against weather, water, and wear and tear by men and animals. Pitching should be constructed at right angles to the slope, so that it may be safe against sliding.

After the embankment is constructed, at least one season should be allowed to elapse, for proper consolidation before the construction of pitching. The stones used should be roughly hewed to fit in firmly in their places. They should be laid with their broadest face downwards and firmly bedded on a layer of muram or gravel (See Fig. 93).

EARTHEN DAM SHOWING STONE PITCHING

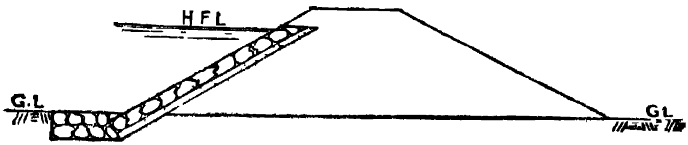


Fig. 93

This bedding is called *gravel backing.* The thickness of gravel backing is usually 6 in. to 9 in., and it is formed on the earthen slope and well consolidated before the pitching work is commenced. No pinning should be allowed between stones, and all hollows and inequalities should be completely filled in with small chips. The top-most course should be horizontal and laid in one level line throughout the length of the embankment. The bottom most course should be finished off with a flat apron to prevent slipping. (See Fig. 93)

Other types of pitching. In addition to the usual rough stone pitching for the water side slope of an earthen dam, some other types as frequently used are noted below.

(1) *Brick pitching:* This is only used where stones are not available cheaply. This is inferior to stone pitching. Only one brick is placed for each course, either as a header or a stretcher. This is to prevent sliding when two bricks are used.

(2) *Concrete pitching:* Where big stones are not easily available, and cement or lime is cheap, this type is adopted. The one danger or defect in this type is, that the concrete usually

racks. This is due to the earthwork below it settling, which cannot be prevented.

The above two are generally considered to be fairly watertight; but for a pitching to be more watertight, the one noted below is advised.

(3) *Pitching either pointed or grouted with mortar or concrete* : The ordinary pitching contains a good quantity of voids. The voids are first filled with big jelly and then with smaller chips and the exterior surface is pointed with mortar; or in the alternative, concrete grouting is poured into the crevices to fill up all the voids which makes the pitching watertight. This method is more costly, but very effective.

(4) One other method used for the river embankment or sometimes for bank embankments in South India is called *Reed Revetment*. The method of construction as given by Buckley is as follows :—

" The reed is one which grows in marshy lands and has numerous joints, at each of which is an eye from which a shoot will spring, if the reed is buried in the ground. A layer of reeds is first placed horizontally on the slope of the embankment with the root ends inward; along the centre of this a fascine, made of the reeds, is placed parallel to the axis of the embankments; the rest

GRASS REED REVETMENT

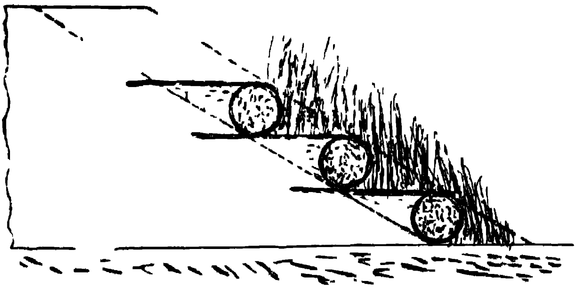


Fig. 94.

of the layer is folded over it, thus forming a step on the slope. The step is covered with soil, and other steps of reeds are formed above it in the same manner. The whole is then watered until the eyes in the reeds throw out shoots. Eventually there rises a forest

of reeds 10–12 ft. high which forms an effective barrier to the action of the waves." (See Fig. 94)

21. Core Wall: *Object*: The object of a core wall is to provide a watertight wall to intercept infiltration from the front or water-side of the dam to the rear.

22. Location of Core Wall: There are various theories regarding the location of the core wall.

The English Engineers recommend the location of the core at the centre of the base. The American theory is that it should be located at the front. The Indian Engineers do not much depend on, or give importance to, the core wall at all. They construct the whole dam of a homogeneous and compact material which prevents any infiltration. Even then, for deep earthen embankments, they provide a core wall of puddle or masonry or both, as circumstances demand.

23. Merits and Demerits of the Core Wall at the Centre: *The advantages of having a core wall at the centre of the dam section are*:—

(1) The core wall is protected from the direct action of water in the reservoir.

(2) It is not exposed to the action of weather.

(3) It is prevented from being perforated by vermin.

The disadvantages are:

(1) The wall itself may settle.

(2) There may be damage to the core wall due to the settlement of the adjoining earthwork.

(3) The core wall cannot be inspected.

Puddle wall or core wall in front:

This acts as a lining in front, and prevents percolation once for all, from entering the dam or embankment.

Advantages:—

(1) The infilteration is prevented at the source itself.

(2) The settlement, if any, is gradual and regular.

(3) It can be easily inspected.

Disadvantages :—

(1) It is exposed to all the dangers, such as, water, weather and vermin.

(2) It cannot stand the usual slope of the dam

(3) The cost is prohibitive.

N. B. The English Engineers prefer a central puddle core, because they get good clay at cheap cost, and the section adopted there is such that the bund at the sides may be of any material.

24. Types of Core Walls :—(as per materials used)

The different types of core walls according to the materials used are

(1) Clay puddle wall

(2) Masonry core wall

(3) Concrete core wall

(1) **Clay puddle wall:** *Excavation of trench* :—A trench for the puddle wall shall be excavated along the centre line of the embankment, until the ground under the dam attains a level of 2 ft. above the F. S. L. The sides of the trench may be cut vertical; if the soil cannot stand it, this slope may be made as steep as it can stand, without shoring, which should be resorted to, when necessary. The usual slope given is 1/4 in soil and 1/8 in rock.

SECTION OF PUDDLE WALL AS PER RANKINE'S FORMULA

$t = 1/2\,b$ or $2/3\,b$
$b = 1/3\ h$

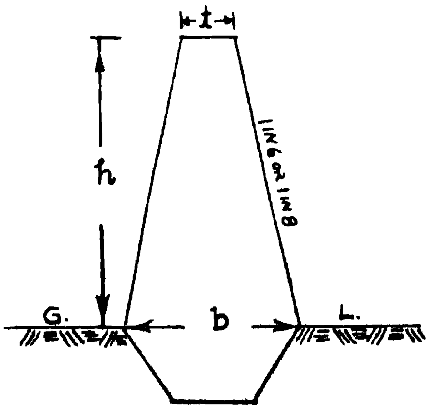


Fig. 95.

Puddle: The trench for the puddle is to be filled with good clay made out of clean, tough and *retentive clay* of the best quality available near the site: the most suitable clay is of the description used for tile making.

The clay before being used should be worked up into puddle by turning it over and over again with a *mumti*, watering it, and allowing it to be trodden by men, so that it may become one plastic, homogeneous mass. This puddle should be neither too wet nor too dry.

Section of the clay puddle wall: The top width of the puddle wall as given by Rankine is 1/2 to 2/3 of the breadth of the base: the width of the base is 1/3 the height. See Fig. 95. The slope given to the sides above ground level is 1/6 to 1/8, and below the ground level it is 1/4 to 1/3. The top level of the puddle wall is kept at about 1 ft. above H. F. L. and 2 or 3 ft. below the top of the embankment. See Fig. 96.

PUDDLE WALL: GENERAL SECTION

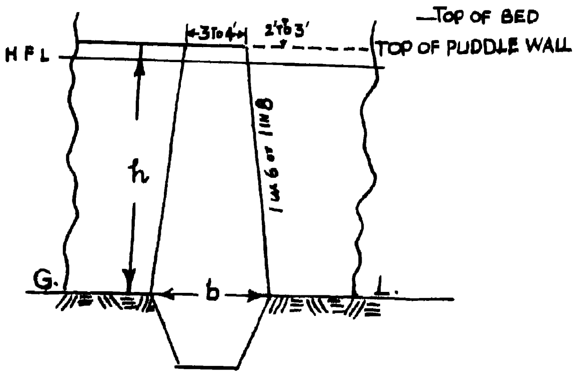


Fig. 96.

Laying the puddle: The bottom of the trench should be cleaned before filling in is commenced. All the inequalities in the foundation should be filled in with balls of puddle thrown and packed well.

The layer of the puddle should not generally exceed 9 in. in thickness. It should be kept as level and as uniform as possible.

The puddle should never be allowed to become dry. If by chance it dries, and cracks are found on the surface, it should be dug out and redone. When there is no work due to holidays, a special cooly should be employed to see that the puddle is kept wet; it is therefore advisible to cover the top surface with wet earth or grass, when the day's work is finished. The sides of the puddle wall should be filled in with selected clay or good material, and well consolidated.

Ramming: The surface of the puddle should be thoroughly consolidated with iron rammers or ponners. Before a new layer is put, the old layer should be lightly sprinkled with water and thorough ramming done. The puddle filling is to be carried to a height of 2 ft. above ground level and joined on with the hearting.

Originally, puddle core walls were being invariably used everywhere. Now after the advent of cement concrete and its use on a large scale, it was found cheaper to use this in place of puddle clay,

MASONRY CORE WALL

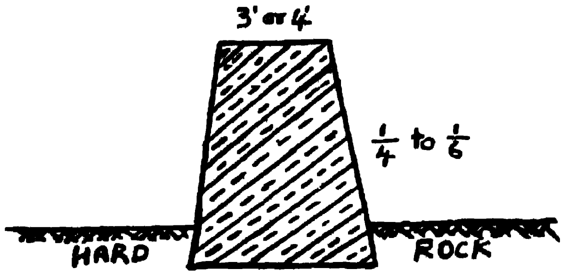


Fig. 97.

and this is also preferred owing to the inferior workmanship sometimes found in the construction of puddle walls.

25. Masonry Core wall: This core wall is used where the soil met with is rocky. The section of the wall used is shown in Fig. 97. Here the two faces are gently battered and sometimes built in steps so that when the earthwork settles, there may be good connection between the two.

Advantages: (1) When there is a leak in the embankment, the clay puddle may be washed away, but the masonry core wall

being situated on solid rock cannot so easily be damaged.

(i) This permits of the construction of a sluice at its junction, with safety.

(ii) It gives the greatest strength to the embankment as it can resist the erosive action of water.

(iii) It is more watertight than an ordinary clay puddle wall.

Masonry core walls are mostly built as only cut-off walls and not as core walls, hence they may be called cut-off walls more correctly.

A cut-off wall is constructed in the foundation bed and its top projects into the impervious section of the earthen dam, just to make a good band with it.

A core wall is constructed to a good height in the body of the dam, even up to F. R. L., to prevent any percolation.

When the depth of the impervious material is more than 30 ft. steel sheet piling is sometimes provided (as in America).

26. Concrete Core wall: When there is no impervious material available nearby and where the only material available is sand and gravel, but rock is available at a reasonable depth (10 to 30 ft. below ground level) a concrete core wall is safer to be used.

Recently core walls of R. C. C. are built in some places especially where the dam is of Hydraulic-fill type, and it is said that the section is made hollow, and wide, to allow one to pass through it for inspection and also for drain which is connected to the general base drain of the dam.

27. Drainage under the dam: Theoretically the parts or sections of a dam should be watertight in front, and become gradually pervious towards the rear. In practice, such a gradual change of material is difficult of construction, but by making the front casing of fairly watertight material, the central core of clay hearting, and the rear portion of coarse material which allows water to percolate, the earthwork of the dam is made as watertight and as self-draining as possible: but further drainage should be provided for, in the shape of base drains.

The *base drains* generally consist of surface drains. The surface of the ground at the rear of the downstream toe is dressed for a width of 30 ft. to an inclination of about 1 in 10. The drain

is cut at the end of this slope with a good fall sufficient to carry the rainfall running off the slope of the drain. The surface drain is constructed in sections of not more than 300 ft. in length. separated from the next drain by an unexcavated strip (say 10 ft. wide) to prevent the formation of scours in the channel. The flow down in these drains should be deviated from the dam, by small outfall gutters and allowed to discharge some distance below the dam.

Downstream Drains: Just downstream and clear of the surface drain, there should be a parallel downstream drain, having a base width of 5 ft. and side slope as slanting as practicable, and with a depth of 10 to 15 ft. This should be preferably carried down to at least 1 ft. below the surface of unfissured rock, (if it exists within a moderate depth.) The trench excavated for the drain should have continuous bedfall and should be filled up to 3 ft. below ground level, with quarry spoil and dry material got from the excavation of the waste weir. This should be covered by 1 ft. of coarse sand and fine gravel, and finally finished off with ft. of soil which may project slightly above the ground level (See Fig. 98). This drain will intercept and pass off any percola-

DOWNSTREAM DRAIN SECTION

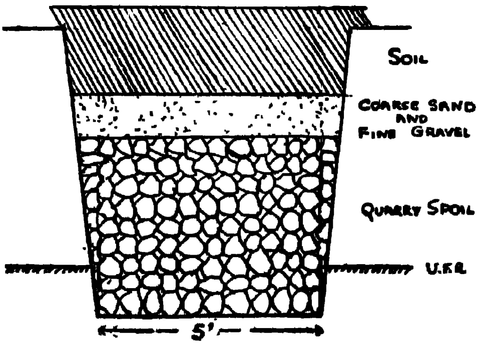


Fig. 98.

tion arising from the puddle trench, and prevent it from soddening the base of the dam. Further it will have a decided effect in

reducing the salt efflorescence which damages agricultural lands. In conjunction with the downstream drains, foundation drains are also provided below the downstream facing of the dam. They are filled with boulders, rubbles, etc. and they are connected with the former at intervals by cross drains.

Cross and Longitudinal base drains: This system is generally adopted in all dams. See Fig. 99. There should be one longitudinal drain close to and parallel to the puddle trench and cross drains from it at intervals of 50 ft. If the bottom width of the dam is considerable, or the subsoil pervious additional longitudinal drains may be provided between the drain behind the puddle trench and the rear toe. The longitudinal and cross drains should join at the same level and the latter should be laid with a bed fall of not less than 1 in 100. The drain should be filled with irregularly shaped rubble or chips about 4 in. in diameter and covered on the top; (it should be covered on the

DRAINAGE AT BASE OF AN EARTHEN DAM

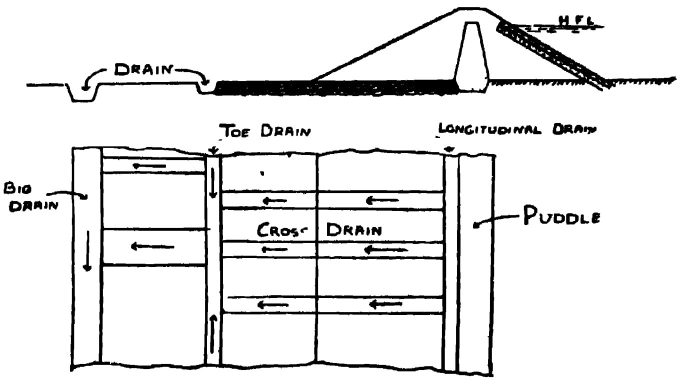


Fig. 99.

sides also if any portion of the drain is in embankment). On the top of this, quarry refuse, small stones, and turf sods should be laid to prevent the earth from being washed into the drains and choking them. See Fig. 100. When expense is justifiable, the bottom of the drain may be made of concrete, and the sides, of rubble masonry and slabbed over on the top.

CROSS BASE DRAIN

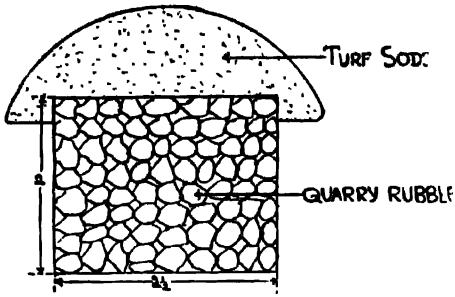


Fig. 100.

Another type of drain in common use is given in Fig. 101.

In hard muram or rock the drains may be cut into the ground and merely covered over with a slab.

CROSS JELLY DRAIN

1. **Big Quarry Spalls**
2. **Small Chips**
3. **Fine Material**
4. **Slab or Tile Covering**

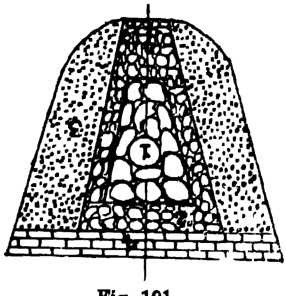


Fig. 101.

The objection urged aganist this system of drainage is that the cross drains at intervals will drain the whole base irregularly and thus tend to produce unequal settlement.

Another system proposed is that a series of small puddle trenches parallel to and upstream of the main central one, is excavated and filled with similar materials as the central trench. On the downstream side of the dam is a similar series of trenches. But these are filled with porous material, like rubble, which drain uniformly and thoroughly the whole overlying part of the dam. They should be excavated in this continuous section, say 300 ft. long, falling uniformly to the downstream ends, where they should

be connected by cross drains of similar sections, which should be carried out of the dam, and continued in covered flat vault leading to the downstream end which will form their outfall.

28. Failure of an Earthen Dam: The failure of an Earthen Dam, like any other structure, is generally due to bad construction and also ineffective maintenance. The defect in the construction is mostly due to the design, and the bad maintenance is due to the neglect on the part of the people concerned.

The earthen dam cannot be designed mathematically like a masonry dam. It is only by the past experience in dams, that a general emperical formula or a practical profile is arrived at. A good many factors, such as the materials used, the water used for the consolidation, the method of consolidation and the general design of the profile, all contribute to the strength of the structure. If any one of these is not properly attended to, there is bound to be some trouble.

The common causes of failure of an earthen Dam are

(1) (Floodwater) Overtopping the bund.
(2) Slipping.
(3) Percolation and leakage.
(4) Erosion.
(5) Defective construction and maintenance.

Taking a good number of failures (in America) of earthen dams or bunds, the causes are noted below in percentage.

(1) Overtopping 35%
(2) Slipping 10%
(3) Percolation and leakage 25%
(4) Erosion and defective construction and maintenance 30%.

(1) *Overtopping*:—The failure on account of overtopping is maximum. It is clear, that the design of the dam in these cases is not properly made. The overtopping is due to, either insufficient waterway for the weir (or surplus work) or insufficient height of the free board. Hence the design should be properly made to provide the necessary area for the flood discharge. Regarding the free board, it may be that the original design does not provide for the required height of the free board, or the maintenance work in keeping the top level of the bund to the standard, may not have been properly attended to.

(2) *Slipping*:—A slip in an earthen bund will occur, only when the slope is steeper than what can be retained by the friction and cohesion of the particles of the soil comprising it. So it is essential that the correct quality and proportions of soils should be used for the construction of an earthen dam.

The slip may occur either on the upstream slope or the downstream slope. To prevent the slip, the front side is given a flat slope and also protected (as this is to be watertight) with rough stone pitching. If the water level in the reservoir is lowered rapidly, the front face has a tendency to slip and generally these slips are local.

With regard to the downstream slope, it should be so designed, that it will not become saturated with water. It should be given a flatter slope than the angle of repose.

(3) *Percolation and leakage*: This percolation may be under the foundation or through the bund proper. The leakage, when it occurs, carries with it particles of soil, forming tunnels or caves in the bund. Eventually, due to the formation of the cave, a slip may occur.

The percolation is due to the porosity of the soil of the dam. Hence the more compact an embankment is rolled, the less will be the percolation.

Percolation below the dam: This percolation can be prevented by the provision of a puddle core wall, and to help the free

EARTHEN DAM WITH CUT-OFF WALLS SHOWING LINE OF PERCOLATION

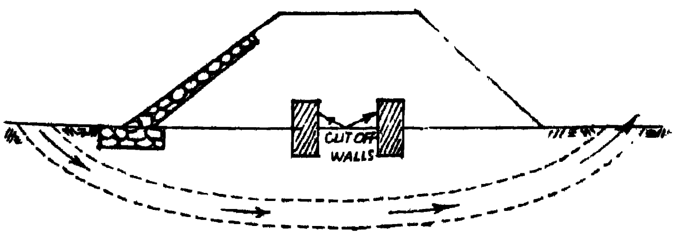


Fig. 102.

drainage of the water collecting at the base of the dam, cross jelly

drains and a longitudinal drain at the toe are constructed and these prevent the slip.

The percolation under the dam may be in the form of what is called *piping*. This is due to insufficient length of the path of percolation, due to defective design. (To lengthen the path of percolation, cut-off walls or stop walls are provided as shown in Fig. 102).

Leakage: Leakages occur through (i) cavities or tunnels formed by burrowing by animals, or (ii) roots of trees which have decayed, or (iii) defective out-let pipes or conduits.

The remedies suggested for the above are:

(i) To provide, for the slopes and the top of the dam, a layer of some material which the burrowing animals cannot penetrate, or to provide greater top widths and greater free boards.

(ii) To maintain the slope clear of all trees, and, when any existing tree dies, to remove its roots completely.

(iii) For the conduits or pipe-outlets, a proper bedding and proper protection *all round* should be given, and the joints specially attended to, and made watertight.

N. B. Failures due to leakage are more in number than those due to percolation.

(4) *Erosion*: Erosion is mostly due to the action of waves, sometimes it is due to rain or wind.

From the above action, the slopes become steep and the upper layers slip, and thus the top level is lowered, and this leads to the overtopping of the flood water and the failure of the dam.

To resist the wave action, stone revetment or pitching is done to the inner or water face of the bund. As a safety measure against the wave action, the height of free board is generally made larger in places where waves predominate. The height of wave in a reservoir is given by the formula

$$H = 2{\cdot}5 + 1{\cdot}5\sqrt{F} - \sqrt[4]{F}$$

where F is the fetch of waves in miles and H is the height in feet; over and above this, allowing also for a margin, the free board is fixed.

(5) *Defective construction and maintenance*: Many earthen dams really fail on account of either (a) Defective con-

struction or (b) Ineffective maintenance, but generally one of the four above mentioned causes is attributed.

(a) *Defective construction*: (i) The consolidation of the bund may not be satisfactory and the water may percolate or leak, and the bund may fail. During the construction of the embankment, if for some time the work is suspended and again resumed, the old surface should be well picked and made irregular to provide for proper bond between the old and the new, as otherwise there is a chance of leak between the old surface which has become hard and smooth and the new layer. (ii) The quality of materials used may not be to specifications. There may be a good amount of soluble materials, which dissolving in water, may create a cavity, and ultimately cause a slip in the bund. (iii) There may be imperfect packing and bonding of the puddle along the outlet pipe in the bund, which may lead to a leak and consequent failure. (iv) The outlet pipe not being properly bedded and protected all round, might crack owing to the superimposed weight. This may result in a leak and failure of the dam. (v) Proper investigation of foundation might not have been done. Hard soil might be seen at the top, but just a little lower down, there may be soft and slushy material which will allow the foundation bed to be saturated and the bund to be sunk, resulting in its failure. In the foundation itself all perishable materials such as stumps, vegetation, roots, etc., should be completely removed and proper bond between the foundation and the embankment provided for.

For example, in the Ashti dam, (India) the foundation was laid on clay and as a consequence, it became very plastic and the downstream side became over-saturated and caused the slip, the rear toe bulging upwards. This could have been avoided, if proper drainage at the rear had been provided in the first instance.

(b) *Ineffective maintenance*:—Regarding maintenance, if the top level is not maintained properly (flood waters will overflow) and if the slopes are not properly maintained, there may be erosion or burrowing of holes by animals. If the base drainage is not properly attended to, there may be choking up of the drains, and consequent stagnation of water, resulting in slips. All these could be avoided if proper attention is paid in time to maintenance work. "A stitch in time saves nine".

Questions

1. When would you adopt an earthen dam in preference to a masonry dam ?

Sketch a typical section of an earthen dam to store a maximum of 50 feet of water. Add a plan and show the drainage arrangement for its downstream portion. Briefly explain your proposals.

(Mysore University, April 1954)

2. Write short note on: Revetments of Earthen Dams.

(Mysore University, September 1954)

3. Explain with sketches the different methods adopted for the final closure of the river portion of a new earthen dam. Draw a typical cross-section in the river portion of an earthen dam 60 ft. high, assuming that rock is met with, at a depth of 5 ft. below the river bed level and explain the object of each of the components of the bund shown in cross section.

(Mysore University, September 1952)

4. (a) What are the practical criteria to be considered in the design of earthen dams ?

(b) What is the theory involved in their design ?

(c) A major tank is discharging over-full on its weir, and it is feared that the bund may be overtopped any moment; also there is a bad leak in the bund. What are the precautionary measures you undertake to save the bund ?

(Mysore University, April 1952)

5. Explain the causes of failure of earthen dams and state briefly the precautions to be taken in their construction.

Draw a plan and cross section of an earthen dam suitable for a storage of 50 feet depth of water and show clearly the revetment, puddle core, saturation gradient, free board, drainage arrangements, etc. The top width of the dam may be assumed to be 12 feet and the depth of foundation 10 feet below the river bed.

(Mysore University, March 1951)

6. An earthen dam 60 feet in height is to be constructed across a stream whose bed consists of a layer of sand 5 feet in depth overlying hard soil Suitable soil is available near the site of bund. Give neat sketches of your design for the bund. Write out a set of specifications for the earthwork tu the bund.

(Mysore University, September 1950)

7. (a) Describe the relative merits of earthen and masonry dams for reservoirs.

(b) Draw a typical cross section of an earthen dam across a perennial stream to impound 50 ft. depth of water above the river bed portion where 12 ft. of sand is found to overlie hard rocky strata.

(A. M. I. E., November 1953)

8. Draw a detail cross section of an earthen dam, assuming a maximum depth of 50 ft. of water behind the dam. How much free board would you allow and how would you calculate the dead and live storage of the reservoir ?

(A. M. I. E., November 1952)

9. Draw a typical cross section of an earthen dam across a perennial stream to impound 60 ft. depth of water above the river bed portion where 10 ft. of sand is found to overlie a hard rocky bed.

(A. M. I. E., November 1951)

10. Give a dimensioned sketch of a section suitable for an earthen bund with a puddle core to hold 35 ft. depth of water. The surface soil is sandy loam to 3 ft. depth with hard, stiff clay below. Drainage arrangements proposed if any should be clearly indicated.

(A. M. I. E., May 1951)

11. What are the causes of failure of an earthen dam (Rolled fill type) ? How can these failures be averted ? Discuss the importance of soil mechanism in the design and construction of an earthen dam.

(Gujarat Univ. April 1953)

12. Write a note on the stability of soils used for the construction of an Earthen Dam (Rolled fill type).

(Gujarat Univ. 1953.)

13. When is the drainage of an Earthen dam necessary and how is it achieved ? What is meant by the saturation gradient of an earthen dam and what is the effect of core wall on it ?

(Gujarat Univ. 1953)

14. Distinguish clearly between the following : Hydraulic fill type and dry fill type earth dams.

15. An earthen dam is proposed to be constructed across a valley. Test pits excavated at the site and at the gorge indicate the following results :—

0′—20′	...	...	Sanday Soils
20′—30′	...	...	Soft muram
35′ to 45′	...	...	Hard muram
45′ and below	...	...	Hard Rock.

Give a typical section of the dam 120 ft. high above the ground level at the gorge portion with all the necessary details; soft muram, hard muram, and black cotton soil are available for construction. Make suitable assumption whenever required.

(Bombay Univ. 1953)

16. Under what conditions would it be preferable to construct an Earthen Dam in preference to a Masonry or Concrete Dam ?

Describe the principal features of design of earthen dams.

(Bombay Univ. April 1949)

17. An earthen dam is to be constructed to store water up to a height of 60 ft. above the river bed. The data available in respect of the material for construction of the dam are as follows :

Dry Density : 118 lbs./c. ft. Saturated weight : 130 lbs./c. ft. Submerged Weight : 70 lbs./c. ft. Moist Weight : 120 lbs./c. ft. Cohesion : 450 lbs./sq. ft.

Trial Pit Data :—Soil–2 ft., Soft muram–3 ft., Hard muram–2 ft., Rock below. Design the section and test the stability of the u/s slope for the worst condition.

(Poona Univ. April 1954)

18. Describe the main principle underlying the design of the earthen dam.

What are the causes of slips occurring in earthen dams ? How can these be prevented ?

(Poona Univ. April 1954)

19. Describe in detail the method of construction of an earthen dam bringing out particularly the special points which require attention to ensure sound construction.

(Poona Univ. November 1953)

20 Draw the cross section of an earthen dam with the following data :

(1) Depth of water at F. S. L. 50 ft. above the lowest level of the bed of the river.
(2) Flood lift 5 ft.
(3) Rock not within reasonable depth.
(4) Good core and hearting materials available.
(5) The top width 15 ft. A berm on downstream 20 ft. wide for wheeled traffic.
(6) Fetch of water in the direction of the wind 6 miles
(a) Provide necessary protection and also arrangements for drainage.
(b) Show in part the drainage arrangement.
(c) What is the function of filter in an earthen dam and where is it located?

(Poona Univ. Oct. 52)

21. In how many ways is an earthen dam likely to fail? Describe the tests you would make on the soil and explain how these tests help to guard against possible failure.

(Poona Univ. 1952)

22. Write Short Notes on:

(a) Flow net, (b) Line of Saturation.

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

23. State in brief the criteria for the design of an 'Earth Dam'. Discuss in detail various methods of drainage and other functions.

Write short notes on: Positive or Partial cut-off for an earthen dam.

(Bombay Univ. B. E. Civil (New Exam.) Nov. 1954)

24. An earthen dam is made up of silt (K = 0·7 × 10 − 3 ft./minute). It rests upon an impervious ground. Its cross-section is as follows :—

Height of maximum water level over ground level 105 ft. Free board 10 ft. Top width 50 ft. Side slopes 3 : 1. A filter bed 5 ft. deep consisting of coarse sand and gravel (k = 1·26 ft./min.) resting on G. L. starts from a point 425 ft. away from U/S toe and extends up to D/S toe. The depth of tail water is one foot. Work out the position of all important points on the 'phreatic line' and show the 'phreatic line' on a cross-section of the dam. (Not to scale).

Discuss very briefly the correctness or otherwise of the given cross-section with reasons.

(Bom. Univ. B. E. Civil (New Exam.) April 1955)

25. What are the causes of failure of tank bunds? State the precautions usually taken to guard against them.

Draw a plan and cross-section of an earthen dam suitable for storage of 40 ft. depth of water and show clearly, the revetment, puddle core, saturation gradient, free board, drainage arrangement, etc. The top width of the dam may be assumed to be 12 ft. the depth of foundation 10 ft. below the river bed.

(Mys. Univ. B. E. Civil April 1955)

26. Write short notes on (a) Composite dams, (b) Breaching section, (c) Flood gap.

(Mysore Univ. B. E. Civil, Sept. 1955)

27. How do you account for the common types of failures observed in earth dams? Explain what tests on soil swell help to forestall such failures.

What are the different methods of controlling seepage through the foundations of an earth dam?

(Bom. Univ. B. E. Civil (New Exam.) Oct. 1955)

28. Write explanatory notes on (a) Reserved filter dams in earthen dams, (b) Casagrande's method of tracing the seepage line in homogeneous earth dam over impervious soil.

State in brief the criteria for design of an earth dam.

What is the utility of each of the following in an earth dam (a) A core-wall, (b) A cut-off wall, (c) A blanket, (d) A rock toe.

What materials would you use for each of them? Give reasons.

(Bom. Univ. B. E. Civil Oct. 1956)

29. How do earthen dams fail? Draw a neat dimensioned cross-section of an earthen dam to hold 40 ft. depth of water. Add a plan and show the arrangements you would make for draining its rear portion.

(Mys. Univ. B. E. Civil April 1957)

30. Write short notes on Breaching section of an earthen dam.

(Mys. Univ. B. E. Civil April 1957)

31. (a) Draw a neat dimensioned Cross Section of an Earthen Dam at gorge, showing foundations on rock. Maximum height of Dam 60 feet.

(b) Why is careful attention necessary to deal with 'Percolation' and 'Drainage', specially in the case of an Earthen Dam?

(c) How is that accomplished in Construction? Give sketches.

(Poona Univ. B. E. Civil April 1957)

32. Bring out the difference between cut-off and core wall.

(Poona Univ. B. E. Civil April 1957)

33. Write explanatory notes on Slip Circle analysis in the design of earthen dams.

(Poona Univ. B. E. Civil April 1957)

34. (a) Under what condition will you recommend the construction of an earthen dam in place of a masonry dam?

(b) What precautions will you take, before and during construction of an earthen dam, to ensure satisfactory work?

(Gujarat Univ. April 1957)

35. Write short notes on Breaching section.

(Gujarat Univ. April 1957)

CHAPTER X

GRAVITY DAMS

1. Theory of Masonry Dams: Masonry dams are designed on some scientific principles unlike the earthen dams.

In designing masonry dams a uniform section, just like an earthen dam, is not given throughout, as this would result in needless waste of material.

The dam acts as a cantilever under the thrust, due to the pressure of water, and the stress increases from top to bottom, and hence thickness at the base should be increased.

In 1853, some French Engineers demonstrated that, in regard to masonry dams, certain definite principles should be followed:—

(1) That the pressure sustained by the masonry or by its foundation should not exceed a safe limit.

(2) That there should be no possibility of any part of the masonry sliding on that below it, or on the foundation.

(3) That it was essential that there should be no tension at any point at any section, or in other words, *the line of resistance should everywhere and under all circumstances lie within the centre third of the dam.*

Rankine considers that the maximum permissible stress should be lower, at the outer than at the inner face of the dam, because the direction of the pressure exerted among the particles of a joint in the masonry, close to the outer face, is necessarily tangential to that face, and unless the face is vertical, the pressure found by ordinary formula, is not the total pressure, but only its normal component.

2. Main Principles: The main principles to be observed in the design of a dam are:

(1) The compressive stress on any area shall not exceed the intensity, i. e., the safe unit pressure which the masonry is capable of bearing.

(2) There should be no tensile stress at any point, least of all on the upstream side. The resultant should pass within the centre third of any horizontal joint.

(3) The resistance to shearing at any horizontal plane, shall be greater than the horizontal thrust of water at the level of the plane.

(4) The thickness of the top must be sufficient to withstand the impact of the floating debris or thrust of ice, and to counteract the effects of contraction and expansion in cold countries.

Note: So far as the stresses are concerned, the top width may be nearly a feather edge, but owing to the above causes as well as to the fact, that a dam is often used as a waste weir also, or for roadway, the top width is made from 10 to 20 ft.

(5) The foundation must be as secure as the superstructure itself and in no part of the structure, there should be any danger of percolation.

Note: This last consideration is never perfectly fulfilled; hence good workmanship is essential.

In designing masonry dams, the distribution of materials should be proportioned to the stresses, allowing for the weight of the superincumbent parts, as well as, for the increasing thrusts of water, as greater depths are reached.

3. Rules (governing the design): For the design of dams, some rules are to be followed strictly, and those given by Creager in his book 'Masonry Dams' are given below.

"RULE 1. GOVERNING THE LOCATION OF THE RESULTANT."

Tension shall not exist in any joint of the dam, under any condition of loading. For dams with rectangular joints, the resultant of all forces acting on the dam above any horizontal joint (including uplift) shall, for full or empty reservoir, intersect the joint within the middle third.

RULE 2. GOVERNING THE INCLINATION OF THE RESULTANT.

The tangent of θ, the angle of inclination with the vertical of the resultant of all forces (including uplift) acting on the dam above any horizontal plane shall be less than the coefficient of friction at that plane.

RULE 3. GOVERNING COMPRESSIVE STRESSES.

The unit vertical and inclined compressive stresses in the dam and the foundation shall not exceed certain prescribed values.

RULE 4. GOVERNING TENSION IN VERTICAL PLANES.

The inclination with the vertical of the down-stream faces of the dam shall be limited to prevent or safely withstand all possible tensile stresses in vertical planes.

RULE 5. GOVERNING THE MARGIN OF SAFETY.

All assumptions of forces acting on the dam shall be unquestionably on the safe side; and all unit stresses adopted in the design shall provide an ample margin against rupture.

RULE 6. GOVERNING DETAILS OF DESIGN AND METHODS OF CONSTRUCTION.

All details shall be adopted, so as not to interfere with the assumptions used in the design; the masonry shall be of a quality suited to the working stresses adopted, and shall be practically watertight and durable.

4. Location of dam: This has been already dealt with in the chapter on Storage Reservoirs; only some special points requiring further attention is noted below.—

(1) The cost of unwatering the foundations, on account of great depth and velocity of water.

(2) Difficulties to be overcome, for the construction of a coffer-dam, if required.

(3) Adequacy of the size of a convenient site near by, for the location of the equipment and camps.

(4) Quantity of silt carried by the stream, which affects the location. *N. B.* In some cases, the silt may fill up the dam in course of time, and the sluices provided are not generally effective in this respect.

5. Dam Foundations: The main points to be considered in the design for the foundation of any dam are :—

(1) Bearing power,

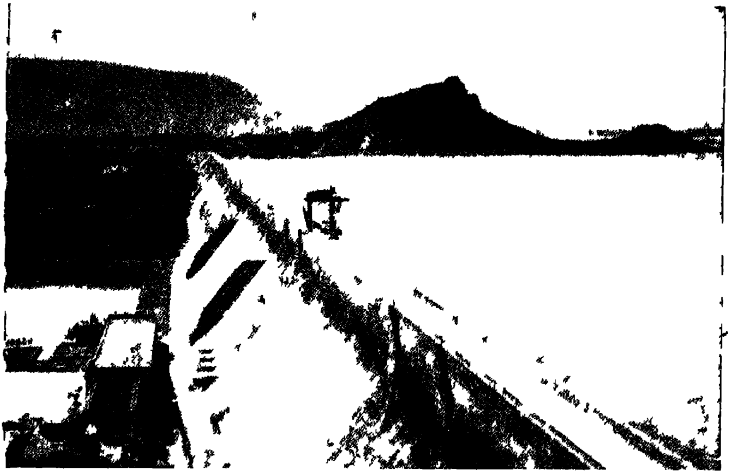
(2) Watertightness or control of seepage,

(3) Prevention or control of upward pressure,

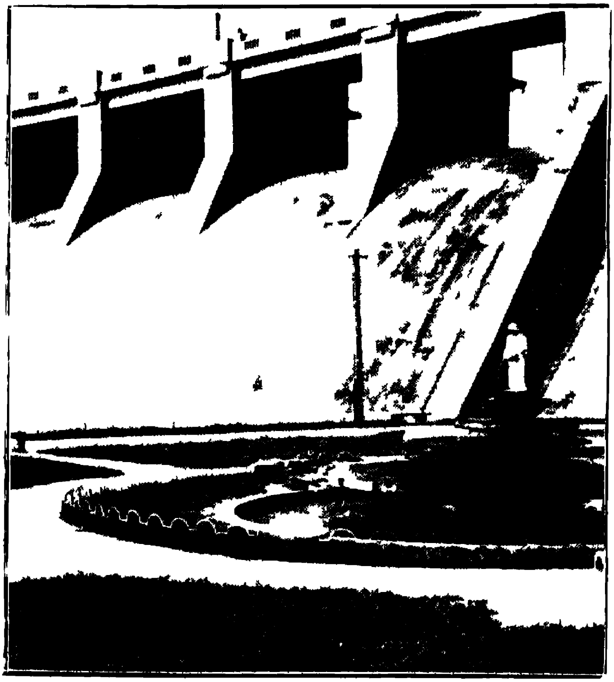
(4) Prevention of the sliding of the dam on its foundation or in the foundation itself, and,

(5) Protection against scour, below the downstream toe or apron.

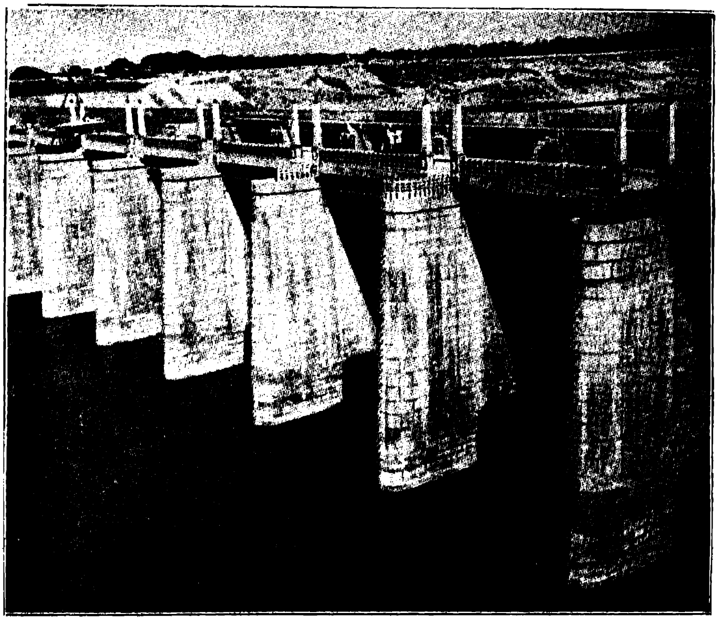
The Mettur Dam



The Malampuzha Dam, Malabar.



A part view of the Nangal Dam (Punjab)



looking from the down stream side showing the concrete parapet, roadway & piers.

The design of the dam depends upon the foundation material. Not only the centre of the foundation length of the dam, i. e., the river portion, but also the ends and the abutments, should be properly attended to, as good many cases of failure of dams have occurred from neglect of proper bond and junction here.

Foundation Material :—(1) *Solid Rock* :—This foundation answers all the requirements for the proper foundation of a dam, but even here, mere surface indications should not be relied upon entirely. Subsurface examinations and thorough preparation of the foundation should be done, before the actual construction is begun.

(2) *Soft Rock* :—This should not be badly fissured. With this material, the bearing power may be alright; but the other requirements may not be fully satisfied; hence special precautions will have to be taken regarding this type of foundation.

Examination of Foundation :—*Rock* :—This should be examined by a geologist as to the bearing power, probability of fissures, faults and any upheavals and disturbances. Soundings, wash-borings, core drillings, and open test-pits should be made, and the proper foundation designed.

Preparation of Foundations :—The surface should be cleared of all rubbish, watered and cleaned. Then neat cement grout must be provided. A layer of 1 to 10 in. thick of rich mortar should be laid, and then the concrete or masonry. Seams of softer material should be cleaned out to sufficient depth, and filled with concrete or grout. If any trimming work is to be done by blasting, it should be done with great care, (short holes and light charges should be used for the first trimming and for final work, only the use of bards or picks or wedges should be made).

For the excavation of foundations, special care should be taken that the foundation bed of good rock is not unnecessarily shattered by blasting. For this purpose, the last one or two feet of excavation is worked using crow bars or wedges. The main principle is " No part of the final foundation should be disturbed from its original position and no stratification should be jarred loose".

Springs in the foundation are not generally plugged before the masonry is started. The drainage is allowed to discharge through pipes, until sufficient masonry is constructed to balance any uplift. The springs are then grouted under pressure.

Grouting : Grouting is now resorted to, in some rock foundations, as a matter of precaution. These grout holes are drilled in two or more parallel lines with the test holes in between. Each grout hole is drilled and grouted, before another hole is drilled. Otherwise the grout is likely to escape through the open holes, instead of being forced into the seams. Neat cement or cement with sand is used for grouting. The mixture is usually 1 of cement to 3 or 5 of water.

* "A method of grouting which seems to have given the best results consists in drilling a primary series of holes in a row under the heel of the dam, from 10 to 15 ft. apart from centres, and of a depth depending upon the nature of the foundation and the head of water to be sustained. The holes are usually about 3 in. in diameter.

"The primary holes are first subjected to water pressure preferably from a tank placed at the same height of the crest of the dam, or a little above it, and the rate of leakage from each hole is recorded. The flow of water also serves to clean out the earth seams in the vicinity of the holes in preparation for the grout, which is to follow.

"After the primary series of holes are grouted, a second series consisting of an equal number of intermediate holes in the same row is drilled, tested and grouted. If necessary, a third series is drilled and treated, thus reducing the spacing to a quarter of that for the first series. The result of a test of any hole is considered an indication of relative tightness of the foundation, between the two adjacent holes previously grouted. The process, therefore, is continued until the tests indicate that the leakage has been reduced to a satisfactory extent.

"The upper ends of holes to be grouted should be provided with threaded pipes with which to make connection to the grouting machine. These pipes must be anchored or weighted to prevent a blow-out during the process of grouting. This is sometimes done by cementing the connecting pipes into the drilled holes or placing them in a concrete cut-off carried far enough into the rock, to provide ample grip. The drilling, in the latter case, is usually done through the pipes. When it is desired that the drilling shall

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* Taken from *Masonry Dams* by Creager.

not interfere with the erection of masonry, the pipes may be carried up along masonry, and the operations of drilling, testing and grouting conducted from whatever elevation the masonry has reached."

Materials: The grout usually consists of neat cement and water, of proportions varying to suit the character of the foundation. For porous rock with fine seams, a mixture as thin as 1 of cement to 8 or 10 of water has been found satisfactory. Where seams are large, and other voids exist a thicker mixture must be used, gradually changing to a thinner mixture as the holes tighten to refusal.

"*Construction*: The grouting operations are usually started from one or two ends of the site, the holes being treated successively. Each hole should be capped, if necessary as soon as it has been grouted, in order that the grout to be forced into the next hole, will not flow back through the completed hole and be wasted, instead of passing on to the ungrouted portion of the foundation."

Keying the Dam proper into the Foundation: A cut-off trench or key-way is excavated along the upstream face of the dam, across the foundation. This trench will not be less than 3 to 4 ft. in depth, depending upon the nature of the foundation, and will be for the full length of the dam. The trench will be excavated vertically with clean square corners. In some cases, one or more additional key-ways are provided within the first 1/3 of the dam. If the bed is of horizontal layer, the key-ways should be carried to sufficient depth to cut off any seams allowing seepage or sliding. The foundation should be made rough, to provide a good bond and prevent sliding. This keying or anchoring the dam to its foundation is done to guard against sliding or the effects of upward pressure.

The drill holes should be properly placed and the grouting in of the reinforcing bars carried up and tied into the masonry, so that the dam and foundation should be bonded together. At both the upstream and downstream sides, deep cut-off walls and at the downstream toe, a well designed drainage system, should be provided.

Overflow Dams on Soft Foundations: An apron of concrete or rock on the downstream side to prevent erosion should be provided. The apron should be so designed, that the standing wave will always occur well back of the apron, and it should be

protected at the downstream end, by heavy rock paving or riprap. Hydraulic jump pools are very satisfactory. If there is a vertical drop, water cushions may be provided, which are very effective. The depth of water should be 20 to 40 per cent of total depth. Even hard rock is not proof against drops. The water in the seams acts as a wedge, taking the impact of the falling water, and eventually rupturing of the rock will take place.

6. Foundations. *Requisites*: (1) A good foundation should be capable of withstanding the weight over it.

(2) It should be watertight to prevent leakage, and thus reduce uplift.

(3) If the foundation is of rock, it should be clean, so that a good bond may be got with the structure above.

(4) It should be rough enough to reduce sliding.

7. Geological Aspects for the Foundation of Masonry Dams:—(1) *Dams on Solid Rock*:—Though the rock is solid, it may be porous. Porous rocks, such as open textured sandstone, decomposed granite or dolerite and laterite, should not be selected, as there will be considerable percolation at greater depth.

(2) *Dams on Bedded Rock*:—If sandstone is exposed in the upper bed of the stream to any distance, the site should not be selected, as there would be heavy leakage.

(3) *Dams Parallel to the strike of the Bed*:—The length of a dam should be situated parallel to the strike of the bedding, and the foundation laid, to have an apron of an impervious bed, under the edge of the dam. If the bed strikes parallel to the length of the dam, and there is a choice between two sites, one with the bed dipping upstream and the other with the downstream dip, it is preferable to locate the dam on a rock with upstream-side dip.

(4) *Faults*:—A dam should not be built across a fault plane. It may be built on the upstream side of a fault, which crosses a valley parallel to the length of the proposed dam.

One of the sites selected for the dam in the Damodar valley had to be abandoned, because it was found that the impounded water would cover the outcrop of a strong fault for a length of four miles, and considerable leakage would take place from the reservoir into the neighbouring coal fields, which were already subjected to serious water trouble.

For the foundation of any masonry dam, it is not merely the soundness of the rock, but also its bearing capacity and its impermeability that will have to be taken into consideration, as well as any defects met with in the rock formation, such as faults, bedding seams, folds, cavities and buried gorges. These should be specially seen to, and necessary action taken to secure proper foundation.

The following information taken from The Institution of Engineers (India) Journal of June 1950* shows how the foundations of important dams, recently under construction in India, were successfully managed.

The Bhakra Dam (Punjab) : At the Bhakra dam site across the Sutlej, there are clay beds about 40 ft. thick or more running right along the length of the dam downstream of the axis. The designed height of the dam being more than 50 ft., it was necessary to take serious note of the clay stratum. The clay would be dug out for a depth of nearly 80 ft. and backfilled with concrete. (See Fig. 103).

The Peechi Dam (Kerala) ; At the Peechi Dam, 130 ft. high here is a dyke running across the dam width, filled with soft

THE BHAKRA DAM TREATMENT OF FOUNDATION

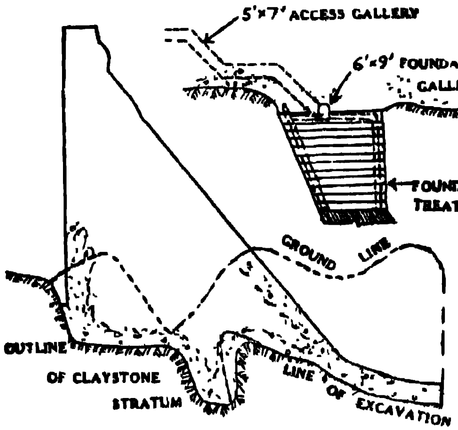


Fig. 103

disintegrated rock. The dyke is 1 ft. wide for some length and

* Extract from Journal of The Institution of Engineers, India, Vol. XXX-IV, July, 1950–' Alternatives in Dam Design and Construction ' by Dr. K. L. Rao.

becomes 20 ft. wide in the rear portion. The soft rock was removed for a depth of 56 ft. and while its width decreased, it did not disappear. The rock was completely porous, and the water was coming up as rapidly as it was being removed. Treatment similar to that of Bhakra would have been sufficient, but the difficulty lay in removing the soft disintegrated rock from the narrow width of only 1 ft., and even this in an inclined direction. Suggestions are made to resort to chemical injections, backfflling and high pressure cement grouting.

The Bhavani Dam (Madras): At the Lower Bhavani (Madras) the rock consisted of hornblende gneiss with ultra basic intrusions giving rise to pockets of talc schist. The width of schist was at places only 1 ft., while at other places it was 10 ft. and more. The vein was dipping upstream at an angle of 50° to 65°, so that it was anticipated that it would do no harm, if it was covered by at least 10 ft. of hard rock. However, it was thought, talc could be avoided altogether, and the alignment of the dam was shifted during execution by 70 ft. to one side and even there, a skirting channel of weathered rock was found. This had to be removed to a depth of nearly 60 ft. below the bed and backfilled with concrete, though in the original design, the anticipated depth of sound rock was taken as only 30 ft.

The Tungabhadra Dam (Mysore and Andhra): The Tungabhadra dam is founded on epidiorite at a depth of 15 ft. below the bed of the river. The rock is dipping upstream and is in slaty formation. A cut-off trench 10 ft. × 10 ft. was dug in the front for 400 ft., but was given up later, as it was considered unnecessary and undesirable. While ordinary consolidation grouting seems to be sufficient in many places, a few cavities are being uncovered, necessitating backfilling. The main source of trouble at this dam site is pegmatite. While the latter presents sheet rock appearance, it is jointed in places, the joints being filled with red clay.

Dr. K. L. Rao gives a description of the foundation tackled here (The Tunga-Bhadra dam).

Taken from the journal of the Institution of Engineers (India) for September 1953.

" The difficulties in the foundation are dealt with in a paper by Sri. M. S. Thirumale Iyangar."

" Epidiorite forms the bed rock in the valley at the site with intrusions of pegmatite closely blended with the bed rock. During actual excavation, a number of cleavage faults between formations in the pegmatites themselves were found. The typical treatment of the fault adopted is shown in Fig. 104. The fault was ¼ in. to

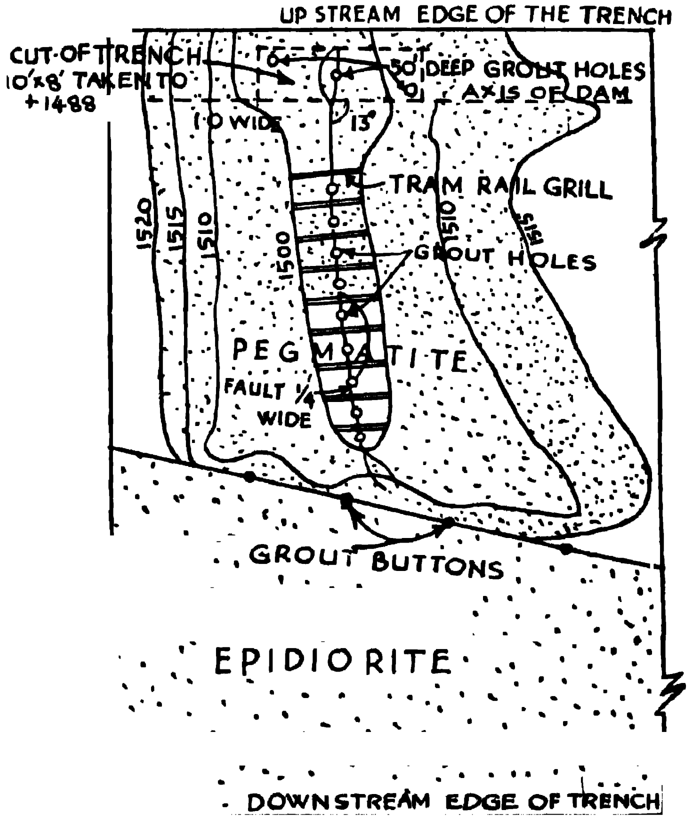


Fig. 104.

1 ft. wide and extended only in the pegmatite formation. The filling material was 70% gravel and silt and 30% clay. The soft material was removed by compressed air and water jets to a depth of 20 ft.

below the general bed rock level. A trench 8 ft. wide and 10 ft. long was dug 30 ft. deeper than the excavated level. Staggered holes 50 ft. deep were put down in the cut-off trench and grouted. Mass concrete was laid in the fissures up to 10 ft. below the rock level with concrete mix of 1:2·38:5 04. Tram rails were laid to span the deep transverse trench at about 20 ft. below the bed level and bars were put in the steep inclined rock slopes on the side. Grout holes were also put down in the mass concrete at 5 ft. centres in level. Finally a system of holes for grouting the contact planes between the concrete and rock was provided, the grouting to be done from the drainage gallery later"

The Mettur Dam (Madras): The ruling rocks in the Mettur dam were charnockites with narrow patches of hornblende gneisses, micaceous schists and pegmatites. Besides joints and potholes in the charnockites, there were bands of mica schist turned into clay, and a number of dykes in the river bed running south of the dam. One of the dykes came just to the toe of the dam at site of the temporary sluices. The broken parts in the dyke were removed and the cavity was grouted.

8. Classification of Dams: Dams may be classified as

(1)	Gravity Dams	Described below
(2)	Arched Dams	Will be narrated in the next chapter
(3)	Earthen Dams	Dealt with already in the last chapter IX
(4)	Timber Dams	Dealt with in Chapter VIII
(5)	Steel Dams, etc.	

GRAVITY DAMS

Gravity Dams: are divided as (a) Solid Dams
(b) Hollow Dams

Solid Dams: These are built of stone or concrete with either hydraulic or cement mortar. They are more permanent than other types.

Hollow Dams are generally built of reinforced cement concrete only.

Advantages of solid gravity dams

(1) They are more permanent than other structures

(2) They require less maintenance charges

(3) The danger of uplift is practically very little.

Disadvantage :—The bearing capacity of the foundation should be enough to withstand the superincumbent weight.

Solid gravity dams are used where an impervious cut-off, at or below the surface of the ground, can be easily got.

Hollow Dams; These are contsructed mostly of R. C. C. They are used where the lift of solid type is expected.

Advantages : (1) They can be constructed sooner than the solid type.

(2) The uplift is very small, due to narrow walls and buttresses (the water under the dam having a duct exit).

(3) The overturning can be reduced, if a good batter is given to the upstream face.

(4) The construction allows the housing of turbines and other apparatus within the dam itself, and thus a good savings in cost.

(5) The structure can be built lighter per square foot of base.

By having spread footings, it can be made to exert less pressure on the foundation; hence, it can be built even on a softer foundation.

9. Classification (*based on materials used* for Construction): The development of the construction of dams was gradual. Up to 1900, rubble masonry was mostly used for the construction of dams. Later on, from 1900 to 1910, cyclopean masonry was used, and recently after about 1922, concrete masonry is being adopted everywhere and on a large scale.

Dams may therefore be classified, as

(1) Rubble Masonry Dams

(2) Cyclopean Masonry Dams

(3) Concrete Masonry Dams.

(1) *Rubble Masonry Dams* :—For the construction, either hydraulic mortar, or cement mortar should be used. The two faces, both front and rear are built of coursed masonry for a thickness of about 5 to 6 ft. and the inside filled with rubble. Both the front and the rear faces are pointed with cement mortar. In some cases,

the front is given a coat of cement plaster instead of mere pointing, to make it more watertight.

This type is better than the ordinary cut-stone masonry which was originally being used and which was more expensive: this has the advantage that it has got a better bond throughout the mass of masonry. The bed is to be made perfect before the stones are placed in position. Spalls should be packed into the mortar bed, where it is thick. All vertical joints should be carefully packed with small rubble and chips. The cut-stone facings should always be one or two courses above the rubble work, to ensure bond between the courses.

One of the best examples of the rubble masonry type of dam is the New Croton Dam. Here big stones varying from 15 to 50 c. ft. in volume were carried by derrick, swung to their places, and lowered on a thick bed of mortar. The New Croton Dam is 1200 ft. long and 18 ft. wide at top and 185 ft. at bottom. The height of the dam is 238 ft.

The Tansa Lake Dam in Bombay is also of this type and the height is 118 ft.

In Mysore, all the masonry dams constructed recently are of this type. For example, the Marikanave Dam, the K. R. S. Dam, the Chamarajasagara Dam (for Bangalore water-supply scheme) and the Hirebaskar Dam (Mahatma Gandhi Hydroelectric scheme).

(2) *Cyclopean Masonry Dam*: Even in this type, the facings are of coursed masonry, but the interior is filled with concrete. This is cheaper than the above type.

This is rubble masonry in which concrete is used in place of mortar, and hence the joints are thicker. As cement of better quality could now be got easily and also cheaper, and skilled masons became costlier, this kind of masonry has replaced the old rubble. The concrete is dumped in the position and spread. On this layer, stones are set by derrick, and, into the spaces between the stones (which are generally 1 to 2 ft. in width), concrete is dumped and chips packed into the interstices. Special care is taken to prevent shearing planes by allowing the big stones to project above, to form a bond with the future work.

Examples of this type are: (i) The Ashokan Dam in New York (America), height 220 ft., with a facing of concrete and (ii) The Kensiko Dam.

(3) *Concrete Masonry Dams*: This is more in use now than the above two types and with the availability of cement at a reasonable cost, this type is found to be the cheapest and the best.

N. B.—Still there are the old methods, the rubble masonry and the cyclopean construction being adopted in some places, owing to local advantages.

Further details of this type (Concrete Dams) are given at the end of the chapter.

10. Forces Acting on the Dam: For the design of dams, the first consideration is the determination of the forces acting on the dam. These forces are

(1) Water Pressure. (2) Earth Pressure.
(3) Ice Pressure. (4) Wind Pressure.
(5) Wave Pressure. (6) Weight of the Dam.
(7) Earthquake Pressure.

A general theory indicating the nature, the amount, the direction and the location of the forces can only be known in each individual case separately, with sound experience and judgment.

(1) *Water Pressure*: In the design of the dam, the weight of water is usually taken as 62·5 lbs. per c. ft.

The weight of water varies for different degrees of temperature. For example, at 32° F it is 62·42; at 75° F it is 62·26 lbs. and at 100° F it is 62 lbs. per c. ft.

The forces acting on an overflow dam are complicated, due to the motion of water.

Dams are subjected to water pressure, not only on the exposed upstream face, but also on their bases and within the masonry itself. These internal pressures produce uplift, and so they also have to be taken into consideration, while designing the dam.

(2) *Earth Pressure*: Rankine's formula

$$P = \frac{wh^2}{2}\left(\frac{1 - \sin\alpha}{1 + \sin\alpha}\right)^2$$

is generally used to find the earth pressure on the dam. This pressure is due to the foundation trench, which is backfilled. This is generally small, and sometimes ignored; but silt accumulated in the bed of the reservoir produces some pressure and this is taken into consideration.

(3) *Ice Pressure :* Ice due to its expansion and contraction exerts a pressure on the dam. The thrust of ice is limited to the crushing strength of ice. It may be anything between 100 and 1000 lbs. per sq. in. This calculation would give about 140000 lbs. per sq. ft. The top layer of the ice solidifies first. As the temperature goes down, the ice sheet becomes thicker by the addition of layers from below, and as this thickness increases, the surface level of water also rises. As soon as the temperature rises during the day, the ice expands and exerts pressure against the dam.

The coefficient of expansion of ice is 0·000027 per degree F and the coefficient of expansion of concrete is 0·000005 or roughly 1/5 of that of ice. Owing to this difference in the coefficients of expansion, there is pressure on the dam.

Atmospheric Pressure : This is taken into consideration, only when the lower *nappe* does not correspond to the profile of the dam, because air is entrapped between the downstream face of the dam and the flowing sheet of water, and a partial vaccum is created.

Note : A dam should always be designed for the maximum flood level, if not, air will be entrapped as the water jet does not adhere to the downstream slope.

(4) *Wind Pressure :* This is not generally taken into consideration in the design of the dams, because the maximum wind pressure is so small, when compared to the load for which the dam is designed, and it acts also with the water load.

If the dam is provided with large sluice gates, then the wind load is taken as 20 to 30 lbs. per sq. ft. Also in hollow dams, for the exposed buttresses, wind pressure is (to be) taken into consideration.

(5) *Wave Pressure :* It is only the upper portion of the dam that is subjected to the action of the waves. To find out the action of waves, different formulæ are given. For example Stevenson's Formula $h = 1{\cdot}5\sqrt{F} + (2{\cdot}5 - \sqrt[4]{F})$ where h is the height of wave in feet and F is the fetch in miles, and

Molitor's Formula $h = 0{\cdot}17\sqrt{FV} + (2{\cdot}5 - \sqrt[4]{F})$ where V is the velocity of wind in miles per hour.

The maximum pressure occurs at one-eighth the height of the spill-way water level. This wave pressure is taken as 150 × the height of the wave.

(For examples on wave pressure, refer to the Book on "Solutions to Problems in Irrigation Engineering" by the authors).

(6) *Weight of Dam*: The weight of the masonry of the dam depends on the materials, (stones and mortar) used for the work. For stone in surki mortar, the weight is taken as 140 lbs. per c. ft.; for ashlar masonry 160 lbs. per c ft. and for rubble 130 to 155 lbs. per c. ft.

Weight of Foundation: To increase the resistance to overturning or sliding, dams are tied down to the rock foundation below, by steel bars grouted into holes and extending into the dam. This theory is not accepted by some, as perfectly appropriate.

(7) *Earthquake Forces:* In the regions where earthquakes occur, dams should be designed to resist the inertia effects caused by earthquakes. This acts on the dam and also on the mass of water. Hence the resultant effect on both should be taken in the design.

11. Requisites for the Stability of Gravity Dams: To satisfy the conditions of stability, a gravity dam should not fail in any one of the following ways.

(1) By overturning on the downstream toe.

(2) By the crushing or the settlement of the foundation

(3) By the crushing of the masonry

(4) By the sliding of the upper portion on the lower horizontal plane

(5) By the opening of any joint, due to tension.

To overcome the above, provision should be made as noted below :—

(1) *Overturning of the Dam:* The resultant due to the water pressure and the weight of the dam must pass through its base. For a dam with rectangular joints, the resultant should fall within the middle third of the base.

(2) *Settlement* :— To prevent settlement, the intensity of pressure must be within the safe bearing capacity of the soil.

(3) *Crushing of Masonry* :—To prevent the crushing of the masonry, the intensity of the maximum compressive stress (at the extreme points of any section) should not exceed the safe limit of the resistance of the masonry.

Compressive Stresses :—

(i)	Rubble masonry	20,000 to 30,000 lbs. per sq. ft.
(ii)	Cut-stone masonry	30,000 to 40,000 „ „ „ „
(iii)	Granite ashlar masonry	50,000 to 60,000 „ „ „ „

These figures include the usual factor of safety. For concrete dams, the compressive stress is 700–1000 lbs./sq. inch with a factor of safety of 4.

For Solid dams, compressive stress at the toe is 1/21 of the Ultimate strength and at the heel it is 1/15th of the Ultimate strength. In the case of the Hollow dams, the compressive stress at both the toe and the heel is 1/15th of the Ultimate strength.

(4) *Sliding*: To prevent sliding, the shear or the sliding resistance of the base must be greater than the horizontal force acting on the section. The horizontal component of the resultant water pressure should be less than the weight of the portion multiplied by the coefficient of sliding friction (Weight of the portion taken includes the weight of the water on its upstream face, if it is sloping. In this case the shear resistance of the material of the dam is neglected).

(a) *Resistance to Sliding (shear neglected)* :—Tan θ (θ is the angle between the vertical and the resultant of all the forces including uplift) acting on the dam above any horizontal plane, shall be less than the allowable coefficient of friction at that plane.

Tan θ should be less than f

$f = 0{\cdot}75$ for Rock Foundation.

Remedies to reduce sliding : If, in a stratified foundation, the upper layers are poorly bonded,

(1) The foundation is taken deep,

(2) A cut-off wall is constructed,

(3) A wedge shaped structure is added at the rear, (see W in *Fig. 84) (a).*

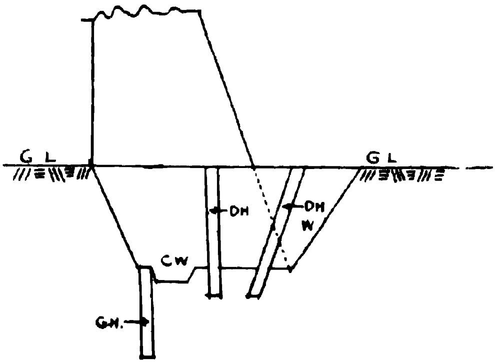
(4) Grout holes are provided and

(5) Drainage holes are driven. See Fig. 105.

If the foundation is safe, but the bonding with the masonry is poor,

(1) The foundation is made sloping towards the pit,

FOUNDATION IN STRATIFIED ROCK



DH–Drainage Hole. GH–Grout Hole CW–Cutoff wall W–Wedge.

Fig. 105.

(2) Proper drainage is provided for,

(3) Grout holes are driven. See Fig. 106.

FOUNDATION TO GIVE A GOOD BONDING WITH THE MASONRY

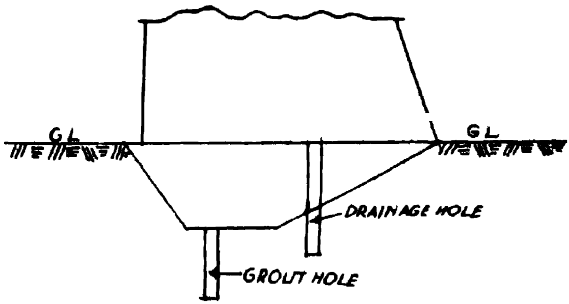


Fig. 106.

(b) *Resistance to sliding (including shear)* :—

The total forces resisting the sliding + ultimate shearing strength of the joint, should be greater than the total *horizontal* force above the joint.

Of the 2 types, cut-off walls and drill holes, the former is the older and was in use for long. The advantages claimed by this

method are (1) It provided perfect inspection of the foundation to be improved.

(2) It is a sure method and also tangible.

Drainage Holes. If it is doubted that the cut-off walls are not very effective, drainage holes or drains are provided to facilitate the easy escape of water which comes out of the cut-off. These holes are drilled at intervals in a row, below the cut-off and given free connection with the tail water. They are drilled just a little above the level of the bottom of the cut-off. If the foundation is to be grouted, the drainage holes should be drilled, after the grouting work is completed.

As the drainage holes serve the purpose of finding the quantity of discharge, they should be located at a higher level than the tail-water.

(5) *Opening of Joints:* To prevent opening of any joint on the face due to tension, the resultant of the forces on any section should strike the base in the middle third, both when the reservoir is full and the reservoir is empty.

N. B.—With this condition, (1) is automatically statisfied.

In addition to the above, the uplift pressure should also be taken into account, as it helps in overturning the dam.

Also by the temperature variation, cracks are sometimes formed and if they are serious, leakages are also formed.

Provision should be made for all these in the design.

N. B.—While making calculations for the design of the dam, the cross section at the deepest bed is taken into consideration, and the stability determined.

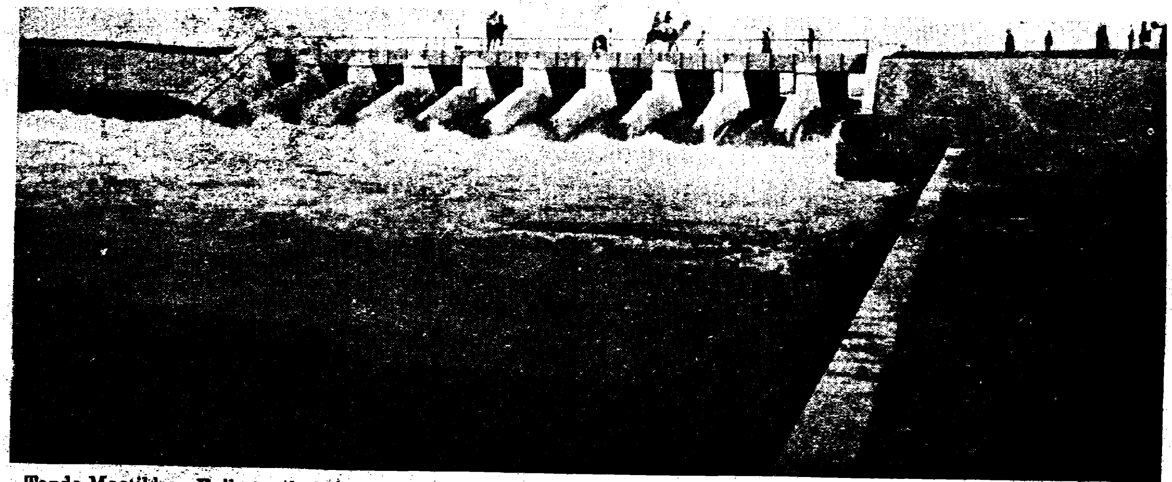
12. Design: The design should give detailed specification, for each work.

General: (1) The stresses should be within the limits allowed.

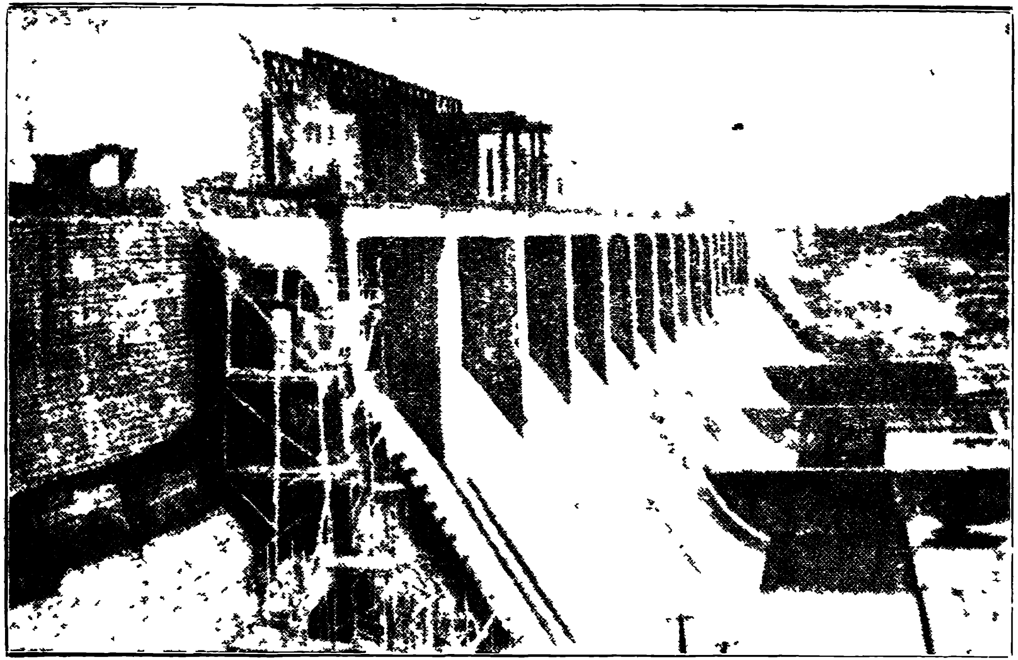
(2) Drains and cut-offs provided for uplift must be carefully designed.

(3) The masonry should be of good quality, to withstand the sanctioned working stress. It should be watertight and durable.

(4) Outlets and spill-ways should be designed to avoid overtopping.



Tando Mastikhan Fall at mile 24 of the Rohri Canal (Sind). Discharge 10,595 cusecs. Width of spans 10 feet. Difference between water levels upstream and downstream is 8.07 feet, and the same between bed levels. Note the vanes constructed from the piers to induce parallel and evenly distributed flow down-stream.



A view of the Tungabhadra Dam. Out of 37 shutters to be erected in the main dam, two have already been installed. Others are ready in the project works

(5) For the main dam itself, proper free board should be provided to prevent overtopping by floods or waves.

(6) Density of masonry should not be less than 135 lbs. per c. ft. In masonry, anything like a smooth and even surface which may encourage either a tendency to sliding or free percolation of water should be avoided.

(7) Masonry should be built as far as possible in uncoursed rubble. Anything approaching horizontal joints should be avoided, as this gives an opportunity for sliding or leakage.

(8) Special care should be taken regarding the quality of mortar. Fat lime should never be used; only hydraulic lime should be used; clean water should be used, because dirty water kills the adhesive property of lime or cement.

(9) Provision should be made for adequate factor of safety.

13. Construction: The following special precautions should be taken in the construction of dams :—

(1) Watertightness in a dam is important; hence the strata of the foundation bed—geological strata—should be studied.

(2) Bunding a dam with the flanks is also very important.

(3) If the dam is of the type of a spillway-dam, the toe should be protected against erosion.

14. Classification: According to the height and size, dams are classified as (1) Low Dams and (2) High Dams.

Low Dams: Low Dams are those :—

(a) In which limitations of allowable stress on masonry do not affect the design.

(b) The basis of design is the elementary profile.

(c) The lines of resistance, reservoir full or empty, must lie within the middle third of each horizontal joint, and as close as possible to the extremities of the outer boundaries of the zone.

High Dams: The design of a High dam is a much more complex process than that of a Low dam.

High dams are those in which (a) The distribution of material in the profile, while following the same rules as those necessary for Low dams, is governed by the limitations in respect to stress (maximum) in masonry.

(b) The width of the base in the lowest layers of a dam profile, should never be increased by introducing flat slopes, as this weakens the toe or heel of the dam. It is not desirable to have any slope flatter than 1:1 on the outer face.

15. Design of a High dam: In the design of a high dam new factors are introduced. A owing to the limitations of stresses, theoretical profile conforming to the standard rules is arrived at, from calculations, and then the final profile, which is a practical modification of the former, is fixed, and this is tested for the stability and stress limit by (a) Calculation and (b) Stress Diagram.

16. Low Dams: The elementary profile of a low dam is known or calculated as above. This is only a theoretical one or as it is called a *Theoretical Profile* (or sometimes called *Elementary Profile* or *Economical Profile*). This cannot be used in practice, because, no dam can have a top width tapering like the apex of a triangle. It should have some width depending upon the requirements.

Top width of dam:—The requirements for the top width of a dam are

(1) A roadway or a foot-way may be required on the top of the dam.

(2) Crest shutters may have to be provided for.

(3) It should be sufficient to withstand the shock of floating bodies.

(4) It should give a neat appearance.

Hence some width is necessary. Mr. Bligh gives this width $a = \sqrt{H}$ where, a is the top width of the dam and H is the height of the dam.

Again the elementary profile assumes the water level to be at the very top or apex of the right angled triangle, which is the section of the elementary profile. Hence provision has to be made for a free board (which is very essential) over the water level.

Free Board: Free board may be defined as the difference in elevation, between the top of the dam and the heighest flood level. It should be adequate to prevent serious damage to the crest of the dam, as a result of wind and wave action.

Free board is classed as :

(1) Gross Free board and (2) Net Free board.

(1) *Gross Free Board*: This is also called *Surcharge.* It is the vertical distance from the crest of the spillway to the top of the dam (See Fig. 107).

FREE BOARD

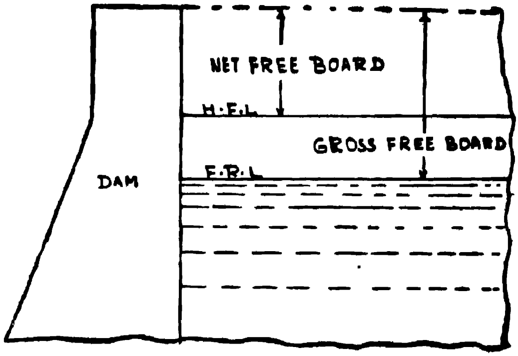


Fig. 107.

(2) *Net Free Board*: This is the vertical distance from the H. F. L. to the top of the dam. Hence Net free board=sum of heights of tides, wind, and wave + margin of safety (Based on judgment).

N. B. In general, *free board* means net free board.

Tide movements are negligible in inland waters (reservoirs); but due to wind, an appreciable piling up of water on the shore is caused particularly in wide and shallow water.

A formula used in this connection *Zuider Zee formula* is

$$S = \frac{V^2 F}{1400\ D} \times \cos A$$

where, S is the set-up or the height in rise in ft. above the F. R. L.

V is the wind velocity in miles per hour.

F is the fetch in miles.

D is the average depth of water in ft.

A is the angle of the wind and the fetch.

The free board should provide for

1 The high flood
2 The wave action
3 Some additional height over the above two.

To know the height of the wave, there are various formulæ in practice (See Appendix) but usually Stevenson's Formula as noted below, is adopted,

$$h = 2{\cdot}5 + 1{\cdot}5\sqrt{F} - \sqrt[4]{F}$$

where h is the height of the waves
and F is the 'fetch' in miles.

Also an additional height of about 3 to 4 ft. is allowed as a matter of safety.

Practical Profile: An extra section is added at the top to provide for the top width and the free board. When this is added the section as now got, is cálled "The practical profile of a low dam". (Vide Fig. 108)

THEORETICAL AND PRACTICAL PROFILE

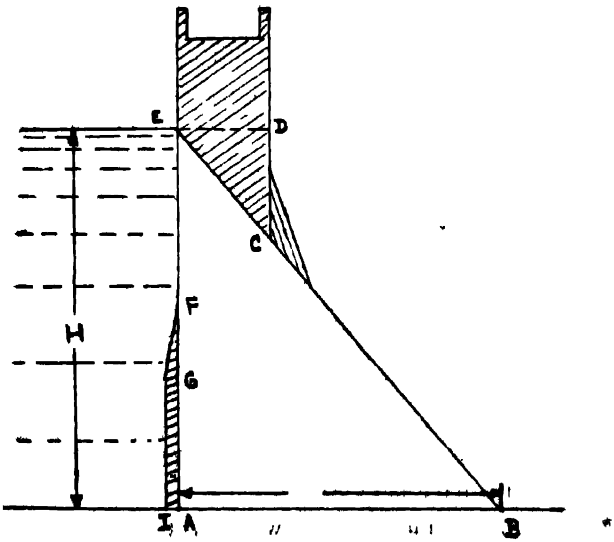


Fig. 108

This addition at the top brings the resultant thrust when the reservoir is empty, just outside the middle third, towards the toe. Hence the front side or the water side of the dam is given an extra section, as shown in the above figure. This is roughly adding 1/16H to the base at the heel. This addition is made not from the very top, but at a depth from the top as shown in the above figure.

17. The Middle Third Rule: Masonry is unsuitable to take up tension, hence dams should be designed to avoid tension.

It is very necessary therefore that resultant pressure should fall within the middle third of the base at every horizontal joint. This constitutes the guiding principle of the design of masonry dams, and satisfies conditions (1) Safe against overturning and (2) Safe against rupture from tension.

This condition must be obtained both when the "reservoir is empty" as well as when the "reservoir is full". (See Fig. 109).

The Minimum profile of a masonry dam: The minimum profile of a masonry dam, which satisfies the above conditions, is a

MIDDLE THIRD RULE

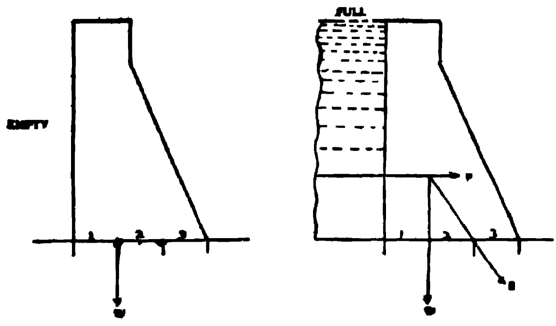


Fig. 109.

Right angled triangle, so that when the reservoir is empty 'R' passes through the inner middle third point and when the reservoir is full, 'R' passes through the outer extreme middle third point. (See Fig. 110).

Consider 1 ft. of the dam,

Weight of dam $= W = \frac{1}{2} H. b. \rho$,

where, ρ is the specific gravity of masonry and the specific gravity of water is 1.

MINIMUM PROFILE OF A MASONRY DAM

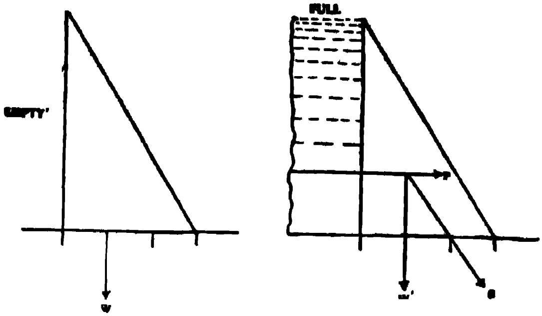


Fig. 110.

P = water pressure $= \frac{1}{2}$ H. H $= \frac{1}{2}$ H^2

Therefore $P/W = \frac{1}{2} H^2 / \frac{1}{2} H.\, b{\cdot}\rho = \dfrac{H}{b\rho}$

But $\dfrac{P}{W} = \dfrac{b}{H}$ [Obtained by taking moments about the point in which the resultant cuts the base]

$\therefore \quad \dfrac{H}{B\rho} = \dfrac{b}{H}$

$\therefore \quad H^2 = b^2\rho$

$\therefore \quad b^2 = H^2/\rho$

$\therefore \quad b = H/\sqrt{\rho}$

This is known as the " Elementary profile " of a masonry dam where the base width is $b = H/\sqrt{\rho}$

This satisfies conditions (1) and (2) i. e., overturning and rupture from tension.

Security against Sliding:—As regards liability to sliding at a joint, this depends upon the friction and cohesion between the surfaces meeting in the joint.

The cohesion between stone and mortar is considerable, but very varying.

The coefficient of friction (tan θ) between dry masonry surfaces is generally taken between 3·67 and 0·75, where α = angle of friction; ('θ' varies between 34° and 37°.)

For safety, the resultant 'R' must not be inclined to the vertical at an angle greater than 34° to 37°; i. e., as in (Fig. 111) it shall not exceed 34° to 37°, according to the class of masonry.

ELEMENTARY PROFILE OF A MASONRY DAM

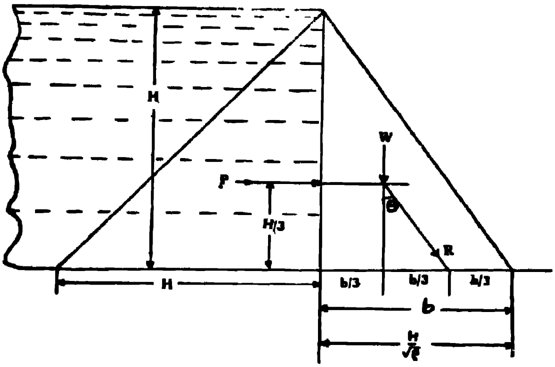


Fig. 111.

In building a dam care should be taken not to have smooth joints but to have at each level an irregular surface, with a large number of stones projecting from it, so as to increase the resistance of the joint to sliding; and if this is done, a gravity dam which fulfils the other conditions of stability will also fulfil this one always (Figs. 112 and 113).

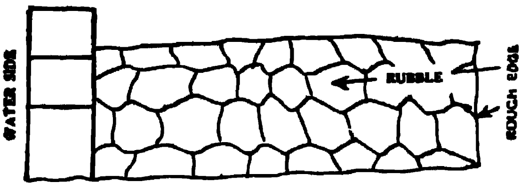
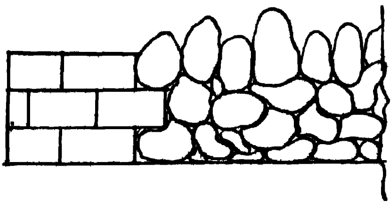


Fig. 112.



The following is taken from Irrigation Manual by Col. Ellis.

18. " Distribution of Pressure in a Masonry Structure "; If S_1 is the mean stress and S is the maximum or minimum intensity of stress, then S is given by the equation $S = S_1\,(1 \pm 6c/b)$, See Fig. 114, where, b is the base width of length of the joint, c is the distance from the centre of this length to the point where the resultant pressure cuts the joint.

DISTRIBUTION OF PRESSURE ON A MASONRY STRUCTURE

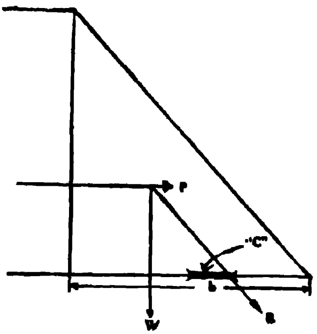


Fig. 114.

(a) If 'R' the resultant passes through one extremity of the middle third of the base $c = 1/6b$, and the maximum pressure is $S_1\left(1 + \frac{b}{b}\right) = 2S_1$ = twice the mean pressure; and the minimum pressure is $= S_1\left(1 - \frac{b}{b}\right) = 0$

(b) If the line of resultant pressure were in the extreme position of cutting an end of the base, in this case $c = b/2$.

$\therefore$ Maximum pressure $= S_1\left(1 + \frac{3b}{b}\right) = 4S_1$

$\therefore$ Minimum pressure $= S_1\left(1 - \frac{3b}{b}\right) = -\,2S_1$

Hence the maximum pressure is four times the mean and is developed at one end of the base and a negative pressure, i. e. tension, having an intensity twice that of mean pressure is developed at the other end of the base.

(c) If the line of resultant pressure cuts the base somewhere within the middle third, say at ¼ of its length from one end of it, then, $c = b/12$.

$\therefore$ Maximum pressure $= S_1\,(1 + b/2b) = 1\tfrac{1}{2}\,S_1$

$\therefore$ Minimum pressure $= S_1\,(1 - b/2b) = 1/2\,S_1$.

(d) If the resultant pressure cuts the base in the centre, then $c = 0$.

$\therefore$ Maximum pressure = Minimum pressure $= S_1$ = Mean pressure.

19. Maximum stress due to the Resultant Pressure: When the reservoir is full, the resultant 'R' acts obliquely to the base.

$\therefore$ The total base subjected to the pressure 'R' may be taken as AC (See Fig. 115).

MAXIMUM STRESS

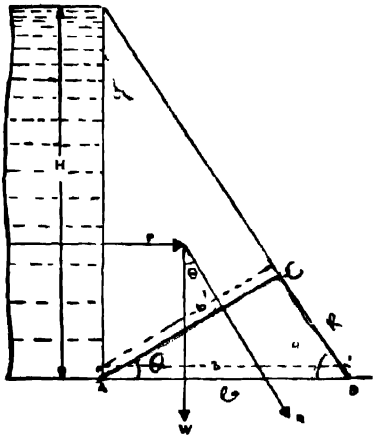


Fig. 115.

$$AC = b' = b.\ \text{Cos}\ \theta$$

$\therefore$ Mean intensity of pressure, at right angles to 'R' when the reservoir is full $= R/AC = R/b' = R/b\ \text{Cos}\ \theta$

$$= \frac{R.\text{Sec}\,\theta}{b} = R'/b.$$

$P = \frac{1}{2}\, w.\, H.\quad H = \frac{1}{2}\, wH^2,$ where w is the weight per c. ft. of water.

$$W = \tfrac{1}{2}\, H.\, b.\, \rho.\, w. = \tfrac{1}{2}\, H.\, \frac{H}{\sqrt{\rho}}.\, \rho.\, w. = \frac{wH^2}{2}\sqrt{\rho} = P\sqrt{\rho}$$

$$\therefore\quad P = \frac{W}{\sqrt{\rho}}$$

$$\text{Now,}\quad R^2 = P^2 + W^2$$

$$\therefore\quad R^2 = \left(\frac{W}{\sqrt{\rho}}\right)^2 + W^2 = W^2\,(\,1 + 1/\rho\,)$$

$$\therefore \quad R = W(\sqrt{1 + 1/\rho}),$$

$$\therefore \quad R'/b = W/b\,(\sqrt{1 + 1/\rho})\,\text{Sec}\,\theta.$$

$\therefore$ The maximum pressure on the base when the reservoir is empty =

$$S_e = 2 \cdot W/b = 2 \cdot \frac{H^2 . w . \sqrt{\rho}}{2} . \frac{\sqrt{\rho}}{H} = H . w . \rho$$

$\therefore$ Maximum pressure on the base when the reservoir is full

$$= S_f = 2 \cdot R/b \,\text{Cos}\,\theta = \frac{2 \cdot R}{b\, W/R} = \frac{2 \cdot W^2 (1 + 1/\rho)}{b \cdot W}$$

$$= \frac{2 \cdot W\,(1 + 1/\rho)}{b} = \frac{2 \cdot H^2 \cdot w \cdot \sqrt{\rho}(1 + 1/\rho)}{2\,H/\sqrt{\rho}}$$

$$= H . w . \rho\,(1 + 1/\rho) = H . w . (\rho + 1)$$

20. Pressure in an Elementary Profile of Specific Gravity 2·25:—

(1) Weight of Elementary Profile

$$= W = \frac{H^2 \cdot w \cdot \sqrt{\rho}}{2} = H^2 \frac{1}{36} \cdot \frac{1}{2} \cdot \sqrt{2 \cdot 25}$$

$$= (H^2/48) \text{ tons,}$$

(2) Resultant pressure (Reservoir full) $= R = W\sqrt{1 + 1/\rho}$

$$= W \cdot \sqrt{1 + 1/2 \cdot 25} = 1 \cdot 2W$$

$$= 1 \cdot 2 \times H^2/48 = (H^2/40) \text{ tons.}$$

(3) Maximum intensity of stress (Reservoir empty)

$$= H . w . \rho . = H \cdot \frac{1}{36} \cdot 2 \cdot 25 = (H/16) \text{ tons.}$$

(4) Maximum intensity of stress (Reservoir Full)

$$= H . w . (\rho + 1) = H \cdot \frac{1}{36} \cdot (2 \cdot 25 + 1) = (H/11 \cdot 1) \text{ tons.}$$

If $\lambda = H_\lambda w(\rho + 1)$ = The maximum stress in tons per sq. foot, then H_λ = Limiting height of the elementary profile

$$= \left(\frac{\lambda}{w(\rho + 1)}\right).$$

If λ = stress in tons per square foot; $\rho = 2{\cdot}25$, then

$$H_\lambda = 11{\cdot}1\,\lambda.$$

If $\lambda = 10$ tons per sq. ft., then $H_\lambda = 111$ ft.

This gives the height of the " Low dam " as per the Elementary profile. If λ varies from 6 to 16 tons per sq. ft., the height varies from $66\frac{2}{3}$ ft. to $177\frac{1}{2}$ ft.

21. Uplift: Ideally, all masonry dams should be built quite watertight. This is impracticable as all the circumstances cannot be favourable. Especially, in large masonry dams water finds its way under the foundation or at the junction of the foundation and the masonry structure, or through cracks (sometimes formed) in the dam.

These are dangerous and produce stresses which have to be met by proper action taken in the matter. The water entering as above, produces what is called uplift pressure. The uplift pressure on any joint may be defined as the sum of the upward pressures due to the hydraulic gradient between the upstream and downstream ends of the joints.

The water causing the uplift may enter through pores of the foundation soil or through imperfectly bonded foundation or through pores in the structure.

N. B.—Water flowing through the pores follows a similar law of decreasing pressure. The uplift causes a reduction in the weight of the structure above it.

The uplift pressures may be graphically illustrated as follows :—

Half Uplift :—Let AB be equal to the base of the dam. See Fig. 116. Assume that the foundation is not fully watertight,

UPLIFT

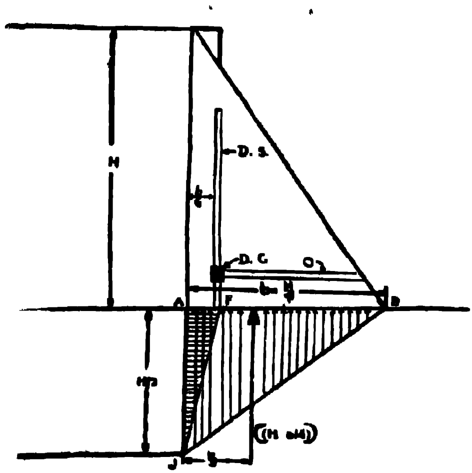


Fig. 116.

about half the upstream head is absorbed as 'head of entry' Therefore uplift head at A is equal to H/2,

and, uplift head at B is equal to zero.

Hence, total uplift head on base is equal to

$$\tfrac{1}{2}\left(\tfrac{1}{2}\mathrm{H}+0\right)\times b\times 1=\frac{\mathrm{H}b}{4}\,.$$

This acts on the base at a distance of $b/3$ from the upstream end.

Full Uplift :—If no "loss of head" is taken, uplift head at A is equal to H and at B is equal to zero.

Therefore, the total uplift head on the base is equal to $\frac{\mathrm{H}b}{2}$.

This is called 'full uplift' due to the head water level.

If now a drain is introduced in the masonry with an outlet at F, at a width of $b/6$ from the upstream face, (See Fig. 116) then the uplift head on the base is dissipated from the point F. The width of the base having uplift is reduced to $b/6$ against the original b. Therefore, uplift head is equal to

$$\frac{1}{6} \times \frac{Hb}{4} = \frac{Hb}{24}$$

(This is for half uplift).

To reduce uplift, provision of drainage is done in the construction of dams. But this has got the effect of increasing the leakage through the dam. In designing the profile of a dam, it is now generally the practice to provide for the force of the uplift also.

Consider the "Elementary profile". (See Fig. 117).

$$\begin{aligned} U &= \text{Uplift Pressure} \\ &= \tfrac{1}{2}(wH + 0)\cdot b\cdot 1 \\ &= \tfrac{1}{2}\, w\, Hb. \end{aligned}$$

UPLIFT ELEMENTARY PROFILE

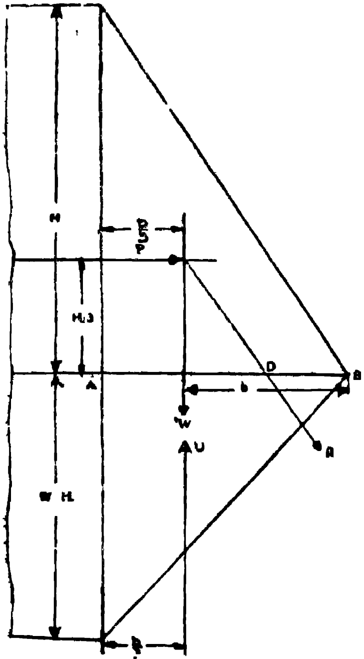


Fig. 117.

Take moments about the point D.

$$P\frac{H}{3} = W\frac{b}{3} - U\frac{b}{3}.$$

But
$$\begin{aligned} W &= \tfrac{1}{2}\, H\cdot b\cdot \rho\cdot w \\ U &= \tfrac{1}{2}\, w\cdot H\cdot b \\ P &= \tfrac{1}{2}\, w\cdot H\cdot H = \tfrac{1}{2}\, w H^2 \end{aligned}$$

Therefore, $\tfrac{1}{2}\, w\, H^2\,\frac{H}{3} = \tfrac{1}{2}\, H\cdot b\cdot\rho\cdot w\cdot\frac{b}{3} - \tfrac{1}{2}\, w\cdot H\cdot b\,\frac{b}{3}$

(Where, ρ = specific gravity of masonry)

Therefore, $H^2 = b^2\rho - b^2 = b^2(\rho - 1)$

Therefore, $b = \dfrac{H}{\sqrt{\rho - 1}}$.

If there was no uplift, $H / \sqrt{\rho} = b$.

$$\therefore \quad H/\sqrt{\rho - 1} \Big/ \frac{H}{\sqrt{\rho}} = \frac{\sqrt{\rho}}{\sqrt{\rho - 1}} = \frac{\sqrt{2\frac{1}{4}}}{\sqrt{1\frac{1}{4}}} = 1{\cdot}36.$$

$\therefore$ Increase in masonry is about one third.

In the elementary profile, full uplift involves an increase of base from $H/\sqrt{\rho}$ to $H/\sqrt{\rho - 1}$, which, with a value of $\rho = 2\frac{1}{4}$, increases the area of the profile by about one third.

This is not economical at all. Therefore the remedial measures as noted are usually adopted, to reduce uplift.

If water stands on the tail water side to a height of "D", then "Uplift" is created by this water also.

Therefore, the portion of masonry below the tail water level may be considered as having "Flotation", and this would provide for 'Uplift'.

UPLIFT (FLOTATION)

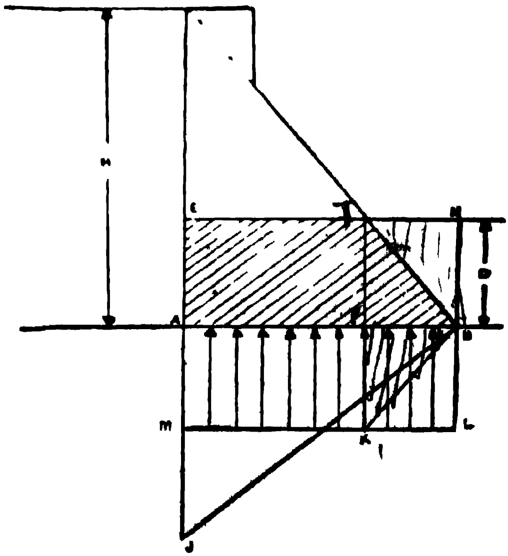


Fig. 118.

In Fig. (118) Full Uplift = AMLB
Flotation = ETBA = AMKB

Balance of Uplift KBL is counterbalanced by TBN the vertical component of the water pressure on the face of the dam.

The term "Flotation" implies the reduction of the weight of the immersed material per c. ft. by the weight of a cubic foot of water. If the specific gravity of the material is "ρ", then the effective weight of masonry subject to "Flotation" per c. ft. is ($\rho - 1$).

The above portion is taken from Col. Ellis—"Irrigation Manual".

23. Coefficient of uplift: When there is uplift, though the water passes through the cleavage, there is a good portion of the base having absolute contact with the lower course or foundation. In any stable dam, the uplift cannot act over the entire area. Hence the total theoretical uplift is reduced.

$$\text{Practical uplift} = c \times \text{Theoretical uplift},$$

where c is the *coefficient of uplift.*

Value of c percentage is 66 or 2/3 for dams on rock foundations and $c = 1$ for earth foundations.

2/3 is the extreme limit, but in practice, a lower value is adopted.

24. Remedies to Reduce Uplift: To reduce uplift, the following remedies are usually adopted :—

(1) Drains in the body and base of the dam are constructed. These reduce the uplift, but in this case there is loss by leakage.

(2) Even in the design of the profile of the dam, provision is made for uplift force. This is done by increasing the section roughly by twenty per cent, which is a very costly item.

(3) The foundation and the masonry are both made impermeable, and for this purpose, cut-off walls are constructed to prevent water passing through cracks or fissures in the rocky bed.

(4) The masonry on the upstream face is made water-tight.

(5) The masonry on the downstream side is made permeable by using a stronger mortar in the front than in the rear.

(6) The dam itself is constructed by using good materials and with good workmanship.

The modern methods of reducing uplift are :—

(1) By providing grout holes. This is cheaper than the cut-off wall.

High and low pressure grouting are now developed to a good extent and are employed in many modern dams.

The front portion of the rock foundation is thoroughly grouted, providing 3 to 7 rows of holes, about 1⅝ inch diameter and varying from 20 to 40 ft. in depth. These holes occupy about 40% of the base width unless the rock is very sound.

At Tungabhadra, where the rock is comparatively sound with no fissure, only 3 rows of holes, spaced 40 ft. centres, are being used for securing watertightness of rock. The pressure employed is also low, being 30 to 70 lbs. per sq. inch. Besides these, deep grout holes are put in at the heel fillet, either vertical or inclined, downstream. These are taken to 75 ft. or more.

In addition to these grout holes, further high pressure grouting is done from the foundation gallery after the dam is completed or till at least sufficient height of masonry is built over the level of the gallery. These holes are drilled deep, roughly equal to ¼ to ½ the height of the dam, and grouted under great pressure of 100 to 500 lbs. per sq. in., and even more in some special cases; the general rule being to allow 1 to 2 lbs. more per sq. inch for every foot depth of the grout hole.

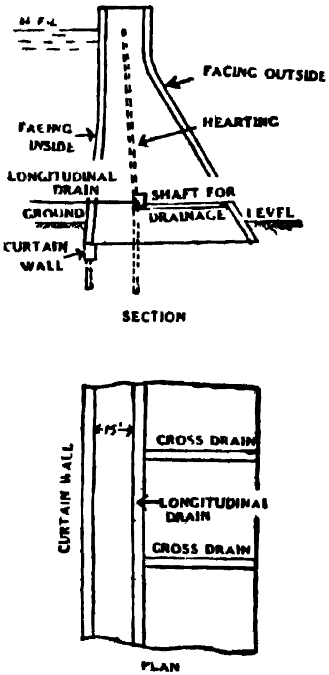


Fig. 119.

At Mettur Dam, no high pressure grouting of rock foundation was done. One line of holes, 1⅝ in. diameter at 40 ft. centres and 3 ft. from upstream edge, and another series of holes 3 in. diameter at 40 ft. centres and 5 ft. 9 in. upstream of the centre line were grouted.

(2) Constructing staunching or cut-off walls in front.

(3) Constructing a drainage gallery. This is done in addition to the cut-off wall or the grout holes.

Foundation galleries (or *Inspection galleries* or *Drainage galleries*) are universally employed now, See Fig. 119.

(a) To enable the drilling of high pressure grout holes to prevent seepage and consequent uplift.

(b) To drill drainage holes for leading off the water, and

(c) For general inspection purposes.

These inspection galleries are of large dimensions, so that as the word implies, one may safely walk through them for inspection purposes. A size of 6 ft. × 10 ft. with a flat top is used in T. V. A. and is found to be satisfactory, See Fig. 120. In Mettur it is 7 ft. × $7\frac{1}{2}$ ft., See Fig. 121.

T. V. A. DRAINAGE GALLERY

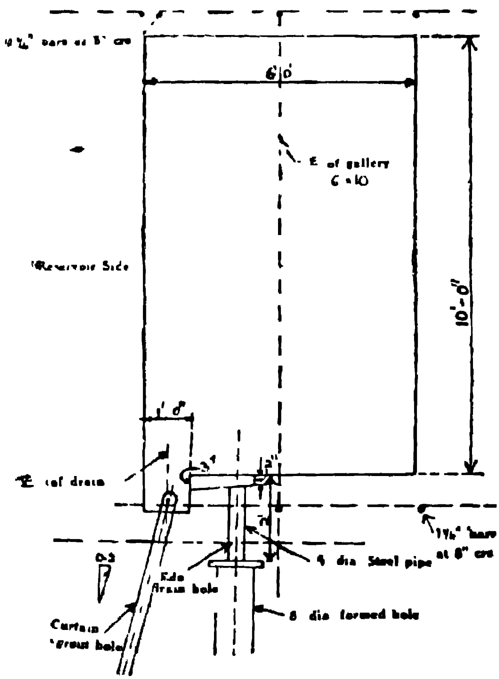


Fig. 120.

This foundation gallery, to be useful, should be located as

METTUR DAM DRAINAGE GALLERY

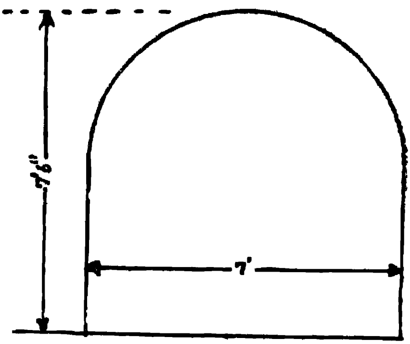


Fig. 121.

near the rock as possible. Hence in the Bhavani Dam, it is provided at 5 ft. above rock level. For the drainage holes, 8 in. to 12 in. diameter holes, at 10 to 40 ft. centres, and 15 to 20 ft. from the upstream face are provided, the water being led into the foundation gallery.

In Mettur, the drainage holes are formed by using pre-cast cement concrete semi-blocks 1 ft. diameter at 20 ft. centres, staggered in rows from the foundation rock to the F. R. L. of the dam.

This, (provision of holes for draining the dam and the rock foundation,) counteracts the damages caused by the leakage of water and helps in the stability of the dam.

(4) The front face of the dam is made watertight.

Watertightness in a dam is very essential as water when it penetrates, increases the stress and reduces the strength of the masonry. Hence to prevent penetration, the face wall should be built of cut-stone and the joints also filled in, with rich portland cement mortar. This is to reduce leakage and thus to keep the downstream face dry.

Various methods of waterproofing the front face are employed.

(i) Three coats of hot pitch and white wash.

(ii) A coat of cement 1 in. thick, and a coat of asphalt.

(iii) Soap and alum wash, three coats, applied at high temperature with a flat brush.

N. B.—On account of waterproofing, no reduction of cross-section should be made in any important dam.

25. Design : For purposes of design, the section of a dam—either of the Non-overflow type or the overflow type—is divided into several sections (generally 7) and each one is designed, in accordance with a different rule or combination of rules.

26. Non-overflow Type : see Fig. 122. This may be divided into 7 zones as stated above.

NON-OVERFLOW TYPE DAM

SKETCH SHOWING THE 7 ZONES

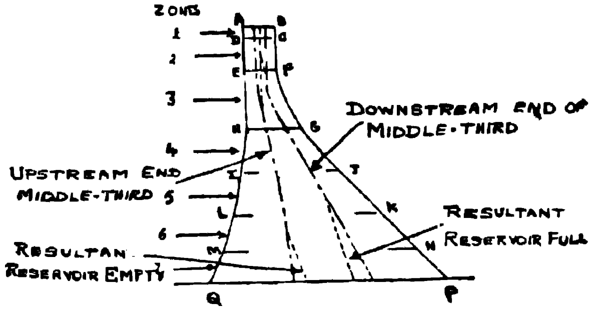


Fig. 122.

Zone I : This zone is the top-most portion of the dam section. The height of this zone is fixed, according to the free board provided, and the top width according to the requirements (Roadway, crest shutter, etc.). If ice is apprehended, this portion begins above the bottom of the ice sheet. As sufficient weight should be provided to prevent this portion from sliding due to ice pressure, the section should be designed to withstand it. See portion ABCD.

Zone II : The resultant, with reservoir full and reservoir empty, should generally lie within the middle third, because the top width is kept sufficiently great. In this zone, both the faces are kept vertical. The depth or height of this zone is fixed until a horizontal plane is got where the resultant-reservoir full intersects the joint at the end of the middle third. See portion CDEQ.

Zone III: In this zone, the downstream face is given a batter, so that the resultant may be located in the proper place, with reservoir full. Here the resultant should still be within the middle third, with reservoir empty. The upstream face should be vertical for such height, where—at the bottom of the section—the resultant with reservoir empty, intersects the upstream end of the middle third. See portion EFGH.

Zone IV. In this section the upstream face begins to have a batter so that the resultant may be located properly, with reservoir empty the position of each face is fixed properly after determining the position of the resultant, with reservoir full or empty, as the case may be, see portion marked G H I J in the fig.

Zone V: This section begins at the place where the inclined pressure is to be limited; i. e. it should not exceed some fixed values. This limit is first reached at the downstream face. The length of the joints should be fixed by the maximum pressure for full reservoir, and the resultant should be fixed properly for the condition reservoir-empty. The resultant, reservoir full, intersects well within the middle third, and for reservoir empty, the resultant continues to intersect at the upstream end of the middle third. See portion I J K L.

Zone VI: The top of this section is fixed by the condition of limiting inclined pressure at the upstream face, and below this level, the slope for the downstream face is fixed by the limiting value of the compressive stress when the reservoir is full, and the upstream face, when the reservoir is empty. This portion is marked K L M N in the Fig. 122.

Zone VII. In this section, both the upstream and the downstream faces are made flatter. This section begins where the value of stress for the downstream face becomes so great, that limiting value of the compressive stress cannot be adopted. As such, the entire design should be revised, to suit all the conditions.

This portion, the lowest, is shown as M N P Q.

27. Overflow Type Dams or Spillway Dams.

Zone I: This is the topmost portion and should be designed to conform to the shape of the falling water. In the top few feet of this section, it is not generally practicable to provide sufficient

weight to insure stability. The resultant-reservoir full-will therefore fall outside the limits of the middle third. Hence there will be no frictional resistance to sliding, if the upper part of the dam should be prevented by the shearing resistance.

If there is ice pressure at the crest, stability by gravity alone is impracticable (because there is very little weight above the ice line).

The resultant cannot be located properly, and so concrete is used to resist tension. A portion of the top height is made thick enough, so that the tensile stress does not exceed 30 lbs. per sq. inch. (If this is not practicable, vertical tension reinforcement is provided near the upstream face). This section stops, where it becomes necessary to locate the resultant at the exact point, see portion A A′ B′ in Fig. 123.

Zone I (a): This is the portion where the resultant can be located properly, but where the resistance to sliding cannot be managed. Above the bottom of this zone, the inclination of the resultant-reservoir empty-makes an angle with the vertical greater than the allowed value, see portion A′ B′ C D in Fig. 123.

OVERFLOW TYPE GRAVITY DAM

SKETCH SHOWING ZONES

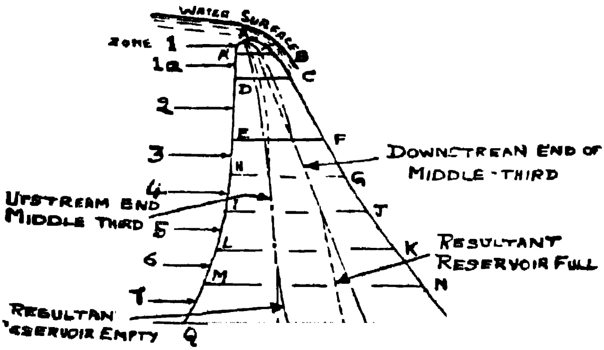


Fig. 123.

Zone II. Here the stability requirements are met, by having a vertical upstream face, and the downstream face to conform to the required overflow shape. The height or depth of this section is fixed by taking into consideration the level at which the resul-

tant for the maximum load conditions will fall exactly at the extreme end of the middle third.

In this section the slope of the downstream face is inclined and not vertical as in the Non-overflow Type, see CDEF.

Zones III to VII are treated similar to those in the other Type.

N. B. For Hollow dams, the arrangement will be different, but the general theory is the same.

28. High Masonry Dams:—*Design of a High Dam*:—The design of a 'high' dam is a much more complex process than that of a 'low' dam.

The section is limited by the maximum stress in masonry. The portion below the 'low' dam profile is widened out, so that the maximum stress approaches the mean stress, and 'R' is towards the centre of the base, when the reservoir is full.

It is undesirable to increase the width of the base in the lowest layers by flat slopes, as this weakens the toe or the heel of the dam. It is not desirable to have any slope flatter than 1:1 on either face.

The theoretical profile of a high dam should always be finalised only after checking it by the graphical method.

HIGH MASONRY DAM

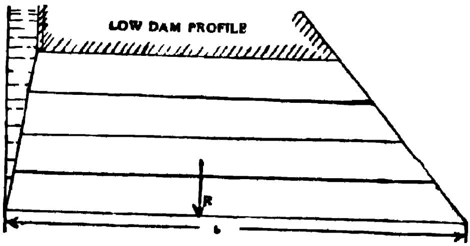


Fig. 124.

Messrs Tudsbery and Brightmore's method of high dam profile:—See Fig. 124.

(1) The top portion up to the limiting depth is the same as for the low dam.

(2) The bottom is inclined out suitably—

λ = Limit stress in masonry, in tons per sq. ft.

H = Depth of Water.

w = Weight of water per c. ft. in tons.

W = the vertical force due to weight of masonry and of any water lying over the sloping section of the upstream face.

b = Width of base.

$$= \sqrt{\frac{w\,H^3}{\lambda}\left(1 + \frac{{}^{1}H^4}{4W^2}\right)}$$

If 'W' be known even approximately, the value of 'b' can be ascertained with close accuracy so as to keep 'λ' within the limit of stress.

A suitable profile may be assumed and tested by stability diagrams, graphically.

By actual graphical method, the stability of the dam should be tested.

Take 1 ft. perpendicular to the figure. Centre of gravity of masonry ABCD is found, as shown in Fig. 125.

DN = NA; CM = MB.

Make AQ = BC; make CS = DA: Join QS and NM. Intersection of the above at 'G' gives the centre of gravity.

SIMPLE TRAPEZOIDAL SECTION–ACTUAL GRAPHICAL METHOD OF TESTING THE STABILITY OF MASONRY DAM.

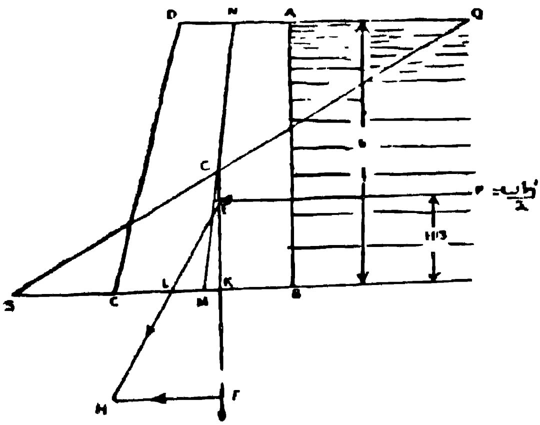


Fig. 125.

For testing stability — $\begin{cases} P\cdot\dfrac{H}{3} = \text{Overturning moment.} \\ W\cdot\dfrac{b}{3} = \text{Moment of resistance.} \end{cases}$

For stability, the overturning moment should always be less than the moment of resistance, with a suitable factor of safety.

30. Molesworth's Formula: Sir Guilford Molesworth, who was the President of the Institution of Civil Engineers, has given a formula to determine the form of a High masonry dam. (See Fig. 129).

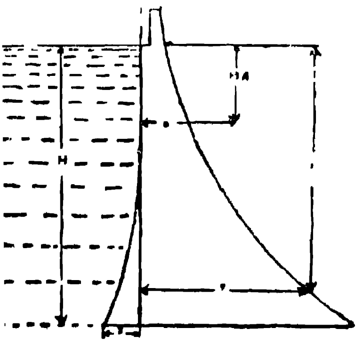


Fig. 129.

$$= \sqrt{\frac{0{\cdot}05\times x^3}{\lambda + (0{\cdot}03\times x)}}$$

$$= 0{\cdot}6\times x, \text{ as a minimum}$$

$$Z = \left(\frac{0{\cdot}09\times x}{\lambda}\right)^4$$

$b = 0{\cdot}4 \times a$ and

$a = Y$ at a depth of 1/4 H

In which, the notations are :—

H is the height of the dam in feet

x is the depth in feet, of any imaginary horizontal plane from the surface of water.

A vertical line being dropped from the top of the upstream face of the dam, then

Y is the ordinate, in feet, from that vertical line to the outer or downstream face of the dam at any depth x.

Z is the ordinate, in feet, from that vertical line to the inner or upstream face.

b is the width of the dam at the top in ft.

λ is the limit of pressure, in tons per sq. ft.

31. Effects of Temperature on Dams: In different portions of a dam, the temperature varies, say from 5 to 15 degrees, from the foundation level to the top of the dam. The down stream

face experiences greater ranges of temperature than the upstream face.

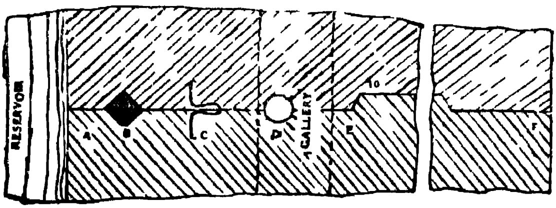
Due to the difference of temperatures, tensile stresses are formed in the body of the dam, and unsightly cracks appear. These cracks open in winter and close in summer.

To prevent the cracks, contraction joints (both longitudinal and transverse) are provided in the body of the dam.

Longitudinal Joints. Concrete is poured in blocks, not wider than 50 ft., so that there may be ready dissipation of the heat of hydration by radiation. The longitudinal joints are carefully grouted before the reservoir is filled, so that it may become monolithic. These joints are always made staggered to ensure discontinuity.

Transverse Joints. These are necessary, wherever the construction is of cement. The spacing between the joints is about 50 to 60 ft. These are normal to the upstream face of the dam and about 60 ft. from centre to centre, along the length of the dam. An R. C. C. column 12 in × 12 in. is inserted near the water face to close the gap of the crack, on the downstream side, See Fig. 130 as shown at B. Even if there is some leakage beyond this point, a stop of annealed copper plate 1/8 in. thick embedded in the masonry (as shown at C) completely stops the leakage; and if there is any further leakage on the downstream side, the vertical drainage shaft (at D in Fig.) connecting it with the drainage

PLAN OF CONTRACTION JOINT USED IN THE METTUR DAM (MADRAS)



A—Upstream face of dam. B—Reinforced concrete (diamond shaped) staunching pillar 12″×12″. C—Copper Strip hair pin. D—12″ diameter drainage shaft to inspection gallery. E—12″ Offset. F—Rear face of the dam.

Fig. 130.

gallery at the base leads off this water. The joint allows free expansion and contraction and at the same time is fairly water-tight.

The heights from which these transverse joints are started depends upon practice. Some authorities start from the foundation level itself, and others from the ground level. In Mettur Dam (Madras) these joints are started at a level of 53 ft. from the rock level.

The transverse joints are intended for contraction. Hence it is better if they are started from a lower level than in Mettur. It is now seen, in the Bhavani and the Tungabhadra dams, that the joints are started even from about 5 ft. above the foundation level.

There are various types of joints in use on dams. One used at Mettur is given in Fig. 130 above, and the other used in Bhavani is given in Fig. 131.

CONTRACTION JOINT, AS USED IN THE BHAVANI DAM (MADRAS)

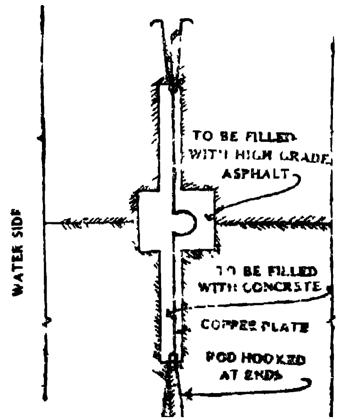


Fig. 131.

CONCRETE DAMS

32. Stresses (and other figures adopted in the design)

Cement weight is 112 lbs. gross in bags
(In America it is 95 lbs. in a bag).

If in barrels it is 376 lbs.

Steel bars are used for reinforcements.

Allowable working stress is 18,000 lbs. in bending, per sq. in. and 13,500 lbs. in compression per sq. in.

Modulus of elasticity is 30×10 lbs. per sq. in.

Strength. Concrete 1 : 2 : 4 has a strength of 2250 lbs. per sq. in. A factor of safety of 3 is allowed.

Hence the working stress for the design is 750 lbs. per sq. in.

For compression with a factor of safety of 3·75, the working stress is 600 lbs. per sq. in.

The tensile strength of concrete is 225 lbs. per sq. in. and the allowable working stress is 75 lbs., with a factor of safety of 3.

The working shear stress is also taken as 75 lbs. per sq. in.

N. B. The strength increases with a greater cement ratio:
with 1 to $1\frac{1}{2}$ to 3 it is 850 lbs. per sq. inch.
with 1 : 1 : 2 it is 975 lbs. per sq. inch.

Generally cement concrete with 1 : 2 : 4 gives a strength of 3000 lbs. per sq. in. in 28 days.

Modular ratio constant used is 15 usually. This ratio changes with the strength of the concrete and is taken as $40000/u$ where u is the ultimate compressive strength. With 1 : 2 : 4 mix, and with u as 2250, the modular ratio becomes 17·8 which is generally taken as 18.

33. Concrete Gravity Dams : *Construction* : One of the most objectionable features in concrete as a structural material is its relatively high coefficient of expansion. To avoid trouble due to expansion and contraction, joints will have to be used at every critical section. For a long time, for dam construction, stones were imbedded in concrete, so as to effect economy in the use of concrete; but now all the ingredients are put through a mixer so that the placing costs are reduced.

The factors affecting the strength of concrete, namely, compaction, grading, batching by weight, and bulking should be carefully controlled, so that good concrete is aimed at. Sand is an important factor affecting the strength of the concrete. River sand cannot always be used directly. If it is well graded it can be readily used in concrete. In some cases, great care will have to

be taken, to prepare the sand according to specification. (The present tendency in U. S. A. is to lessen the cement content in the concrete).

(1) *Mixing concrete*: To get a high degree of uniformity, all the materials should be measured by weight instead of by volume. The mixers should be so placed that the operator is able to see the mixing operation during its progress.

Mixture of concrete: For dam construction, the concrete that is used should be of the desired durability, impermeability and strength with minimum cost. To get the proper mixture, trial mixtures are made until the desired characteristics are got.

In the massive structure of a dam, it is economy that is important. For this purpose, the use of the maximum size of aggregate that is available at site, and that can be handled advantageously through the mixers, is used for the work. For the mixture of concrete, only the absolute minimum quantity of water for the proper placement of the mixture on the spot, should be used.

For multiple arch or slab and buttress type of dams, where the thickness of the members have to be adhered to rigidly, the size of the aggregate is fixed.

N. B.—The water cement ratio is the important factor to control permeability, and any change in this respect is not advised.

Batching and Mixing: For the modern dam construction, batching and mixing plants are in general use. Here the aggregate is delivered to the top of storage bins by belt conveyors, and the cement is then pumped through pipes. The materials descend by gravity through the plant and into the bucket placed below.

The batching plant is generally air-operated and electrically controlled. The tilting mixers are charged from a common collecting store and discharged into the bottom dump buckets, through a central common hopper. This tilting mixer is the only type that satisfactorily handles the mixes.

Transportation and placing: The modern method of transportation is by the trestle and the cableway. These distribute the concrete uniformly over the structure. When a cableway is used, the bottom dump buckets are transported from the mixer to the cableway on cars. The cableway is equipped with a movable tower

at its ends, so that it may serve any point on the dam. When a trestle is used the cars transport the buckets from the mixer along the trestle to the place where it should be deposited, and here a crane picks the bucket off the cars, and the concrete is placed in position.

The consolidation is generally done by vibration with as little segregation as possible. The vibrator is advantageous in retaining the water-cement ratio, because it permits the concrete, which is too dry, to be placed by the hand satisfactorily.

In mass concrete dams, the exterior portions are constructed with more cement than the interior. In such a case, for a depth of 5 ft., normal to the face, the concrete contains more cement proportion than the concrete in the interior of the dam. It is placed simultaneously with the adjacent interior mix, so that the two mixtures will unite in their plastic state, to form an integral mass.

To control the maximum temperature in mass concrete, it is usually poured in 5 ft. lifts. Form-work difficulties increase rapidly with height. Further, in high lifts, water and cement may accumulate at the top of the lift and produce porous and non-durable concrete with a thin band of high water-cement ratio.

(3) *Curing of concrete*: Horizontal surfaces can be cured by applying a blanket of saturated sand, soon after placement is completed. For vertical surfaces, a system of pipes with spray nozzles is arranged at intervals, so that the entire surface is covered with a fine spray. Curing compounds can also be used, but it must be remembered that no curing compound will give the eqivalent of continuous water-curing.

(4) *Joints* :—Joints in a dam are highly undesirable; but in a concrete dam, they are necessary to prevent the formation of cracks, and to permit economical and systematic construction. It should be seen that the concrete immediately below a joint is of normal consistency and there is no accumulation of water. To permit proper bonding of old and new concrete, and the proper bedding of the aggregate in the fresh concrete, ½ in. of mortar is usually applied immediately before the concrete placing commences.

Forms : When forms are necessary to be used for the work, timber is the only material that economically permits the flexibility which is necessary for mass concrete. Sheet steel is not well adapted as it easily wrinkles and buckles.

In the construction of spillways, the slope and bottom surfaces are sometimes screeded. This is not recommended, because this results in the disintegration of the material on the surface, as it is impossible to compact the concrete on the slope, to a durable density.

The construction refinements and the cooling operations used to eliminate or to reduce cracks in the concrete used for dams, have made them outstanding structures.

Necessity for waterproofing: Leakages occur in the dams, and as such, waterproofing has become a necessity. These leakages are due to—

(1) Cold joints between pours,
(2) Settlement or shrinkage cracks,
(3) Careless placement,
(4) Segregation,
(5) Laitance bands at the tops of lifts.

(5) *Temperature control*:—The control of temperature is the most important problem in the construction of a concrete dam. A number of cracks occur due to changes of temperature. Deep cracks are created by high interior temperatures; and surface cracks are caused by the difference of temperature between the surface and the adjoining surface areas. Much attention should be paid to surface cracks, because they are the entering wedges of wholesale disintegration.

Once started, surface cracks may progress clear through the structure.

Cracks can be avoided by,

(a) Allowing a sufficient interval between lifts, and limiting the height of lifts, to say 5 ft. (about),

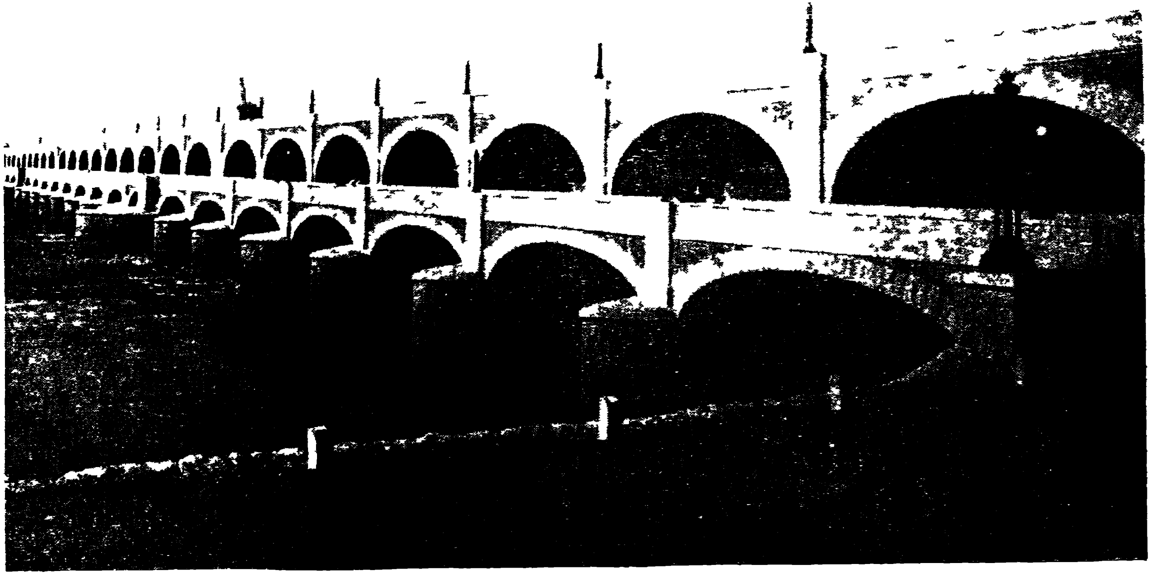
(b) Sprinkling the coarse aggregate and blowing compressed air through it, so that temperature is reduced and moisture content standardised.

(c) Using relatively high cement content for the exterior shell.

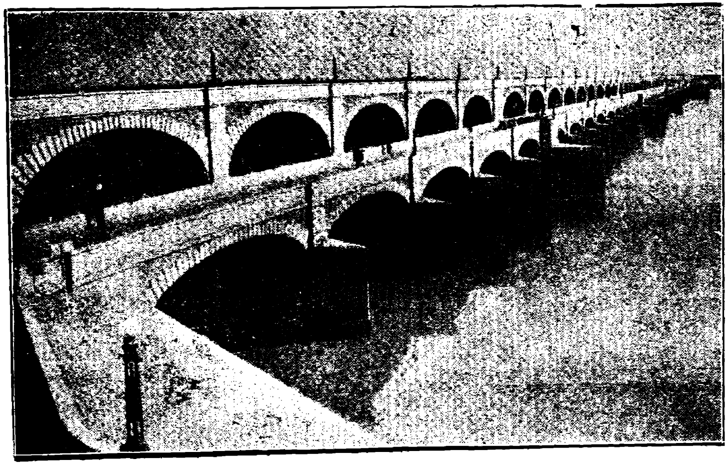
(d) Using low-heat cement.

(e) Cooling the mixing water by refrigeration (including the use of ice).

(f) Circulating cold water through pipes embedded in the concrete.



Downstream view of the Lloyd Barrage (Sind) showing the road bridge as well as the gate bridge. The lamp standards are of exceptionally fine design. The lengthening of the central abutment pier is clearly shown The width of the spans is 60 feet.



The Bhandardara dam in Madhya Pradesh,

(g) Controlling the form removal, so that a large difference of temperature between the surface and adjoining areas does not occur.

To get good, and crackless concrete, effective dissipation and reduction of heat of hydration should be secured. For reduction purposes, puzzolonas (usually powdered brick) are used. For dissipation, refrigeration is adopted. Cooling of concrete in high dams is very essential, to avoid cracking during construction, and to grout the construction joints in the dam. Cooling can be done by circulating refrigerated water through pipes embedded in the concrete at 5 ft. intervals, thus abstracting the heat of hydration from the concrete. Now-a-days, the aggregate is per-cooled, to about 35°F, and by using the mixing water at 30°F., the temperature of the concrete can be controlled to 50° to 60°F.

(6) *Plant layout*.—When concrete is employed, the daily outturn can be as high as 20,000 c. yds. But huge mechanical plants will be necessary; for example, electric shovels, feeders, conveyors, hoppers, electric vibrating feeders, trammel screens, trippers, travelling draglines, derricks, belt conveyors, mixing plants, portable pumps, bucket elevators, trestles, cableways, cantilever cranes, whirleys, hauling cars, crushers, batching plants, etc.

The initial cost of the mechanical equipment required will be considerable and is justified, only if there is an appreciable amount of concrete to the laid. (The cost of crushers, conveyor-belts, and batching plant for pouring concrete at the rate of 250 c. yds. per hour may come to about 35 to 40 lakhs of rupees).

34. Advantages and Disadvantages of Concrete Dams:– The *advantages* of concrete construction for dams are

(1) Machinery can be used for a large amount of work, if the site is easily accessible.

(2) The section of the dam can be made thinner, so that lesser quantity of masonry is required.

(3) The progress of the work is rapid, thus reducing interest charges on the capital invested.

The main *disadvantages* are that concrete dams are more costly unless cement is available nearby. Further the work is likely to be stopped by breakdown of machinery. The maintenance

and servicing of the mechanical plant may present serious problems.

Of the important projects under planning, the spillways for Hirakud, Vaitarna, and Bakhara, are being done in concrete. In the Bakhara dam the great height (680 ft.) and rapidity of construc tion indicate the absolute necessity for the use of concrete in the dam construction.

REINFORCED CONCRETE DAMS

35. Reinforced Concrete Dams:—Another variety of concrete dams is the Reinforced Concrete Dam. Here the section may be made thinner and the work can be finished more rapidly. For the design and construction, it requires careful scrutiny and supervision.

36. Classification:—There are several types of Reinforced Concrete Dams in use. They are usually classified as follows—

(A) *R. C. C. Deck dam or Ambursen type of Dam*:— Fig. 132. This may be either the ordinary storage, non-over-flow dam or an overflow dam. It can be constructed either on soft soil or hard bed; if on the former, the height of the dam should be low. This type is generally used for a wide river with a sandy bed, and also where the height of the dam is not much.

The material or soil on which the dam rests, is the factor on which the design of the foundation is based. If the foundation is of soft soil, drainage from behind the concrete core wall up to the rear toe of the dam should be attended to. If the foundation is of rock, a layer of ordinary cement concrete is enough, but if it is of inferior stuff, concrete slabs should be laid for the full base width of the dam, to take the stress from the butresses.

To prevent percolation below the dam, a cut-off wall is provided, where the bed is rocky, but, if the foundation is permeable, a trench is excavated down to the impermeable soil and the trench filled with cement concrete. If the impermeable soil cannot be met with at a reasonable depth the core wall should be taken to such a depth, that the path of percolation will allow only the safe velocity. It is also advisable to have a layer of watertight material near the face of the slab.

Design and Construction:—Reinforced deck dam consists of R. C. C. face slabs, and buttresses. The slabs hold the water on the upstream side. They are placed inclined to the horizon at an angle of 45° and they are sometimes given a coating of gunite. These slabs rest freely on the buttresses which are of triangular section and are found to be most satisfactory.

The general design should be as for a solid type of gravity dam. The face slab should withstand not only the water load, but also the component of its own dead weight normal to the upstream face, and transmit it to the buttress.

The thickness of the slab should be a minimum of about 12 in. at the top, and increase towards the bottom. As usual with any concrete slab, provision should be made for the expansion and

R. C. C. DECK DAM

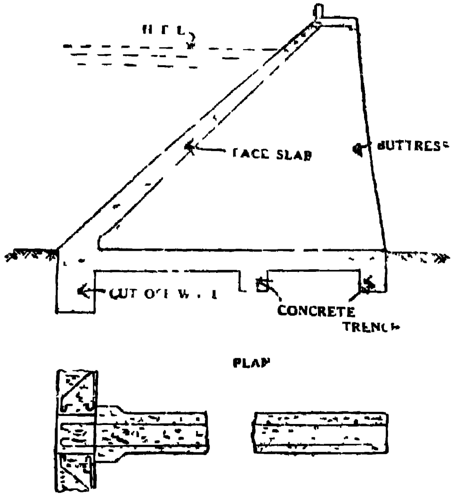


Fig. 132.

contraction of the slabs. The crest slab should be made strong enough to resist the shock from the drift, just as in an overflow dam.

The buttress should be designed not only for its own dead load, but also for half the slabs on either side. The length of the buttress usually adopted is $1\frac{1}{2}$ D to 2 D, where, D is the depth of water. When this is done, tension does not occur. The thickness

of the top of the buttress is kept 12 in. minimum. The buttress walls are constructed about 15 to 20 ft. centre to centre.

The downstream slab cannot be designed by the ordinary formulae. It is usually designed to a thickness of 2/3 the thickness of the slab on the crest. At the downstream toe (as usual with the type of an overflow dam) an upturned bucket is provided. To overcome the vacuum due to the flowing water, etc., usually a few holes are provided in the slabs.

(B) *Hollow Gravity type of R. C. C. Dam*:—If the dam is of the overflow type, a slab is also provided on the downstream face. In this type, the rear is also provided with a slab. That means to say, the slab is continuous from the front toe to the rear toe. This type is called the Hollow Gravity type. In this type, for inspection, an inspection gallery is also provided running from buttress to buttress. (See Fig. 133).

R. C. C. DAM—OVERFLOW TYPE

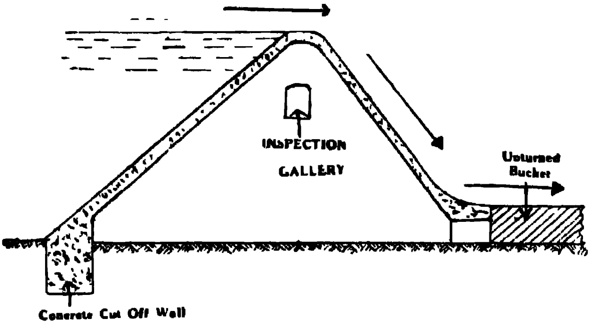


Fig. 133.

(C) *R. C. C. Dome Dam*:—Recently R. C. C. Dome dams are constructed in U. S. A. They are of unique design with multiple domes.

37. Failure of masonry Dams: Most of the failures of masonry dams are not due to the design of the dam proper, and there are cases also where the failures are due to the lack of proper foundation. Examples of some failures are given below.

(1) *The Tygra Dam* (Near Gwalior, Central India): This dam failed on account of lack of proper foundation. The foundation

rock was sandstone 3 ft. thick, interbeded with sheets of shale (clayey sandstone) with low dip towards the downstream.

The foundation here had gone only to a very small depth, $1\frac{1}{2}$ to 2 ft. It had not been properly benched to counteract the downstream dip of the strata; also no water cushion had been provided, to protect the downstream toe of the dam.

In the deepest portion of the river, a breach of 1300 ft. occurred and complete sections, varying from 100 to 200 ft. lengths, were washed down, being pushed bodily and swung round by the force of rushing floods.

The cause of the breach was the upward thrust of water from under the dam, the water finding access beneath the horizontal layers of rock through vertical fissures or dykes.

(2) *The Austin Dam :* In the foundation of this dam, weak horizontal stratified limestone was met with, and in addition, there was fault also in the strata under the base of the dam. The limestone had very little cohesion near the fault, and consequently about 500 ft. length of the dam was removed bodily downstream.

(3) *The Bouzey Dam* : There were fissures in the rocky foundation met with, and also there were veins of micaceous stuff and fissures of clay, which were not removed completely and grouted with cement, before the foundation work was started. Again the foundation did not go deep, as was required. The stresses assumed for masonry were not adequate; the actual stress being more than what was assumed; and hence, the failure of the dam. (The upper portion was swept away for a length of 600 ft.)

General : Many of the failures are due to faulty foundations. The defect seems to be that it is not receiving as much attention as it should, because the engineer is not the designer who generally makes many assumptions in his design and the engineer who actually constructs it may not know them all. As such either a detailed specification of the work should be given or, if possible, the designer should supervise the work.

N. B.—For problems on this chapter, refer to the book " Solutions to the Problems in Irrigation Engineering ", by the authors.

Comparison of Solid and Hollow Gravity Dams:

SOLID	HOLLOW
1. Ordinary labour can easily manage the work.	This requires more skilled labour.
2. Being solid, it is more safe against sabotage.	This is susceptible to sabotage or military attack.
3. This requires more quantity of material per running yard of dam.	This can be managed where the materials are scanty, as it requires less quantity.
4. Turbines and other machinery will have to be installed outside the dam, and hence additional cost.	With the hollow type of dam, turbines, etc., can be easily located inside the dam.
5. The foundation should be strong, as the weight of the dam is more than that of a hollow one.	Distribution of pressure on the foundation is more uniform, as such, even a weak foundation can be managed. (By giving footings, the weight of the foundation can also be reduced).
6. Where the work is of concrete, less form work (practically nil) is required here.	This type requires a good amount of form-work.
Placing concrete is easy, and the cost is less.	Placing of concrete and of the steel reinforcement require special care, and hence the cost is more.
7. In ordinary locations where materials are got cheap and the communication easy, this type is economical.	This type can be used economically only in out-of-the way places, where the cost of the material is high.

Problems on Dams

Problem No. 1.

An earthen dam is formed on a pervious foundation. The top width of the dam is 15 ft. and the front slope is $1\frac{1}{2}$ to 1, and rear slope 2 to 1. The maximum depth of water is 52 ft. and the free board 8 ft. Determine the total seepage through and under the dam assuming the permeability coefficient as $0{\cdot}4 \times 10^{-4}$ c. ft. per minute and the area of the soil through which the flow takes place is 50 sq. ft.

Solution :– The discharge through the dam per ft. width

$$q_D = k\left(\sqrt{d^2 + h^2} - d\right),$$

where d = the width of base $- \; 0{\cdot}7\, m$,

where m is the horizontal projection of the maximum head of water

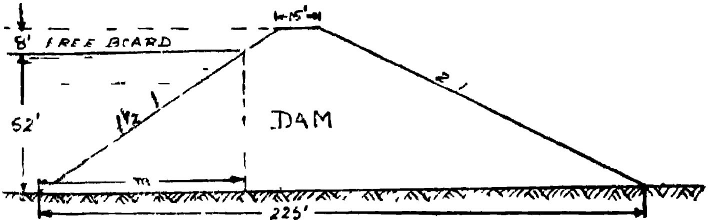


Fig. 134.

Width of base

Front slope portion $= 60 \times 1\frac{1}{2} = 90$ ft.

Top width $= 15$ ft.

Rear slope portion $= 60 \times 2 = 120$

Total $= 225$ ft. (See Fig. 134)

$$d = 225 - 0{\cdot}7 \times (1\tfrac{1}{2} \text{ of } 52) = 225 - 54{\cdot}6 = 170{\cdot}4$$

$$k = 0{\cdot}4 \times 10^{-4} \text{ c. ft. per minute}$$

$$= 0{\cdot}00004 \text{ c. ft. per min.}$$

$$q_D = k\left(\sqrt{d^2 + h^2} - d\right)$$

$$= 0{\cdot}00004\left(\sqrt{170{\cdot}4^2 + 52^2} - 170{\cdot}4\right)$$

$$= 0{\cdot}000308 \text{ c. ft. per min.}$$

Again discharge per ft. width of foundation

$$\therefore \quad q_F = k\left(\frac{h}{l}\right) \times A,$$

where $h = 52$, $l = 225$ & A area of the soil through which the flow takes place is 50 sq. ft.

$$= 0{\cdot}00004 \times \frac{52}{225} \times 50$$

$$= 0{\cdot}000462.$$

The total discharge through the dam and foundation =

$$q_D + q_F = 0{\cdot}000308 + 0{\cdot}000462$$

$$= \mathbf{0{\cdot}00077} \text{ c. ft. per ft. width of dam.}$$

Problem No. 2.

An earthen dam is constructed on a pervious soil 50 ft. in thickness, with an impervious base of length 150 ft., and the depth of water (gross head) on the blanket is 40 ft. Find the length of the impervious blanket, assuming the mean horizontal permeability coefficient of the pervious underground soil is 0·12 per minute, and the percentage of flow under the dam without a blanket is 0·20.

Solution:—The discharge under the dam without a blanket is given by the equation

$$Q = k\left(\frac{h}{b}\right) d,$$

where k is the mean horizontal permeability coefficient,
h is the gross head in feet on impervious upstream blanket,
b is the length of impervious portion of the base of dam,
and d is the depth of the pervious underground in feet.

$\therefore \quad Q = 0{\cdot}12 \cdot (40/150) \cdot \times 50 = 1{\cdot}6$ c. ft. per min. per running foot of dam.

Now, the length of the impervious upstream blanket is given by the formula,

$$l = \frac{(khd - pQb)}{pQ}$$

where p is the percentage of flow under the dam without a blanket

$$\therefore \quad l = \frac{(0{\cdot}12 \times 40 \times 50) - (0{\cdot}20 \times 1{\cdot}6 \times 150)}{0{\cdot}20 \times 1{\cdot}6}$$

= **600 feet.** (See Fig. 135).

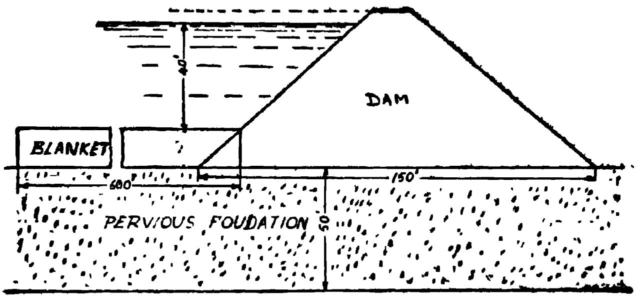


Fig. 135.

Problem No. 3.

An earthen dam has a top width of 20 ft., with side slopes of 2 to 1 and is 70 ft. in depth. It is constructed on a pervious soil which is 50 ft. in thickness; and as such is provided with a blanket in front for a length of 600 ft. (to prevent seepage). Determine the thickness of the blanket, assuming k_1 the average permeability of the underground soil $= 250 \times 10^{-4}$ and k_2 the permeability of the blanket $= 1 \times 10^{-4}$.

Solution:—A formula generally used in this connection is

$$t = \frac{k_2}{k_1} \times \frac{b}{d} \times X \quad \text{where}$$

t is the thickness at a point X from the upstream toe of the blanket,

b is the length of blanket from upstream toe to dam and

d is the depth of underground pervious soil.

$\therefore$ t at a distance $100'$ from the upstream end of blanket (Vide Fig. 136).

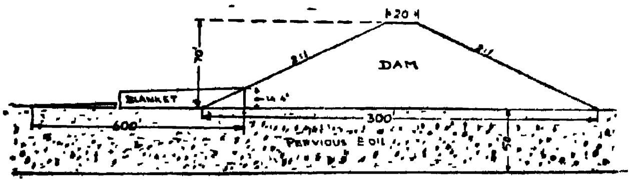


Fig. 136.

Weight of water is taken as 62·4 lbs. per c. ft.).

$$= 22744{\cdot}8 \text{ lbs. wt.}$$

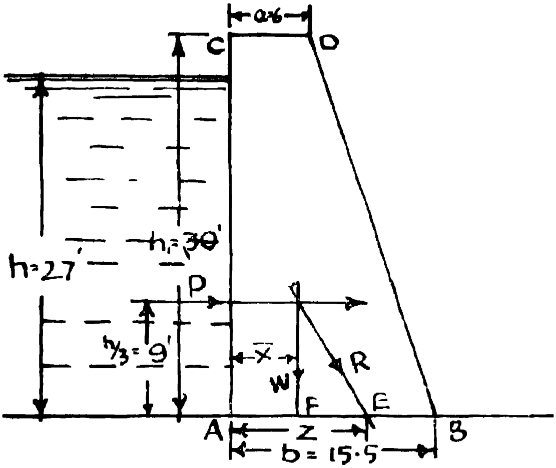


Fig. 138.

The weight $\overline{W}$ acts at a distance $\bar{x}$ from the vertical face as given by the formula,

$$\bar{x} = \frac{a^2 + ab + b^2}{3\,(a+b)} = \frac{b^2 + 6b + 36}{3\,(b+6)} \text{ feet.}$$

The distance from the vertical face of the point of intersection of the resultant of P and W on the base is given by the formula,

$$z = \bar{x} + \frac{h}{3}\cdot\frac{P}{W} = \frac{b^2 + 6b + 3b}{3(b+6)} + \frac{27}{3}\cdot\frac{22744{\cdot}8}{2100\,(6+b)}.$$

To avoid tension on the base, it is assumed $z = 2b/3$.

$$\text{Therefore, } \frac{2b}{3} = \frac{b^2 + 6b + 36}{3\,(b+6)} + \frac{9 \times 22744{\cdot}8}{2100\,(b+6)}$$

$$\therefore \quad 2b\,(b+6) = (b^2+6b+36) + \frac{27 \times 22744{\cdot}8}{2100}$$

$$\therefore \quad b^2 + 6b = 36 + 292{\cdot}43 = 328{\cdot}43$$

$$\therefore \quad (b+3)^2 = 337{\cdot}43$$

$$\therefore \quad b + 3 = 18{\cdot}37 \qquad \therefore \quad b = \mathbf{15{\cdot}37 \text{ feet.}}$$

Again to prevent sliding,

μW should be greater than P (where μ is the coefficient of friction which is taken as 0·6)

$\therefore \quad 0{\cdot}6 \times 2100\,(b + 6) > 22744{\cdot}8$

$\therefore \quad 1260\,(b + 6) > 22744{\cdot}8$

$\therefore \quad (b + 6) > 22744{\cdot}8/1260 = 18{\cdot}05$

$\therefore \quad b$ should be greater than 12·05 ft.

But b should be equal to 15·37 feet as above.

Therefore, the bottom width may be made just greater than 15·37 say **15·5 ft.**

Problem No. 6.

A dam is 400 ft. high and its upstream face is vertical and the width of the base is 296 ft. The top width of the dam is 24 ft. The weight of masonry may be taken as 140 lbs. per c. ft. If the eccentricity of the resultant is 48·73, find the maximum shear stress, also the compressive stress on the downstream face.

Solution: Height of dam = 400 ft. See Fig. 139.

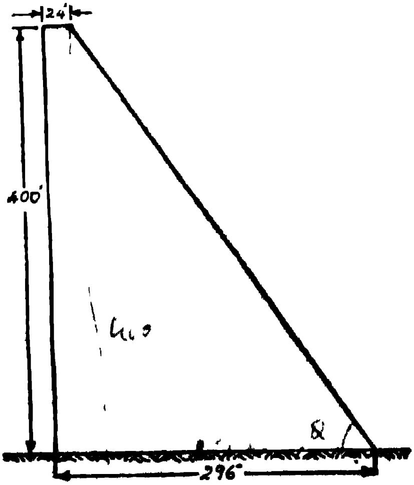


Fig. 139.

Base width of dam = 296 ft.

Top width of the dam = 24 ft.

Area of dam per running ft. $= \dfrac{24 + 296}{2} \times 400$

$= 64{,}000$ sq. ft

Hence, the weight of the dam $= \dfrac{64{,}000 \times 140}{2240}$

$= 4000$ tons.

Maximum vertical compressive stress at downstream toe

$= \dfrac{W}{b}\left(1 + \dfrac{6e}{b}\right) = \dfrac{4000}{296}\left(1 + \dfrac{6 \times 48{\cdot}73}{296}\right)$

$=$ **26·87 Tons/sq. ft.**

Rear batter is 272/400 and so $\cot\theta = 0{\cdot}680$.

or $\theta = 55°47'$ and $\operatorname{cosec}\theta = 1{\cdot}2093$.

Maximum Compressive stress (Principal) $=$ Maximum vertical compressive stress $\times \operatorname{cosec}^2\theta = 26{\cdot}87 \times (1{\cdot}2093)^2$

$=$ **39·25 Tons/sq. ft.**

Shear $= \frac{1}{2} \times$ [Minimum vertical compressive stress $\times \operatorname{cosec}^2\theta$]

$= \dfrac{1}{2}\left\{\dfrac{W}{b}\left(1 - \dfrac{6e}{b}\right)\right\}\operatorname{cosec}^2\theta$

$= \dfrac{1 \times 4000}{2 \times 296}\left(1 - \dfrac{6 \times 48{\cdot}73}{296}\right)\operatorname{cosec}^2 55°47'$

$=$ **0·1198 Tons/sq. ft.**

Problem No. 7.

Find the factors of safety at the base against failure due to (i) Crushing, and (ii) Sliding of a gravity dam of the dimensions as given below :—

Maximum depth of water is 60 ft. and Free board is 4 ft.

Section of dam is rectangular: 12 ft. wide at top for a height of 16 ft. thereafter the U/s face is vertical and the D/s face has a batter of 1:0·75 (Vertical: Horizontal).

Taking the weight of masonry as 160 lbs. per c. ft. and the allowable compressive stress in masonry as 10 tons per sq. ft. and the Coefficient of friction as 0·667. Assume full uplift over half the area

what is the defect of this section ?

Solution:—Referring to Fig. 140,

$FD = 0{\cdot}75 \times EF = 0{\cdot}75\,(64-16)$

$= 0{\cdot}75 \times 48 = 36$ ft.

Base width of the dam = b = CF + FD = 12 + 36 = 48 ft. Let the resultant R meet the base in 0 and let the eccentricity be e.

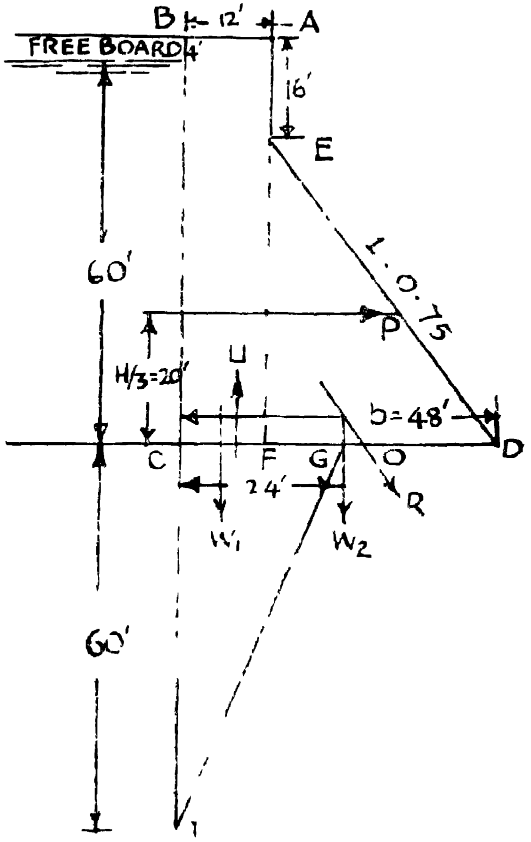


Fig 140.

Now, (1) Horizontal water pressure = $P = \dfrac{62{\cdot}5 \times 60^2}{2}$

$= 112500$ lbs. wt.

Its moment about 0 = $-(112500 \times 60/3)$

$= -2250000$ ft. lbs.

(2) Weight of portion ABCFE = $W_1 = 12 \times 64 \times 160$

$= 122880$ lbs. wt.

Its moment about 0 = 122880 (18 + e) ft. lbs.

(3) Weight of portion DEF = $W_2 = \frac{1}{2} \times 48 \times 36 \times 160$

$= 138240$ lbs. wt.

Its moment about 0 = $138240 \times e$ ft. lbs.

(4) Uplift pressure = $\frac{1}{2} \times 60 \times 48 \times \frac{1}{2} \times 62{\cdot}5$

$= 45000$ lbs. wt.

Its moment about 0 = $- \{ 45000 \times (16 + e) \}$ ft. lbs.

Now sum of the moments

$= 122880 (18 + e) + 138240e - 2250000 - 45000 (16 + e)$

This should be equal to the moment of the resultant R about 0, i. e. the sum of the moments should be equal to zero.

$$\therefore \quad 2250000 = 122880 (18 + e) + 138240e - 45000 (16 + e)$$

$$\therefore \quad 216120 e = 2250000 + 720000 - 2211840 = 758160$$

$$\therefore \quad 758160/216120 = 3{\cdot}51 \text{ ft.}$$

Maximum compressive stress $= \frac{W}{b} \left(1 + \frac{6e}{b}\right)$

Where W is the resultant of downward force $= W_1 + W_2 - U$

$= 122880 + 138240 - 45000 = 216120$ lbs. wt.

$\therefore$ Maximum compressive stress

$$= \frac{216120}{48 \times 2240} \left(1 + \frac{6 \times 3{\cdot}51}{48}\right) \text{ tons/sq. ft.}$$

$= 2{\cdot}8945$ tons per sq. ft.

$\therefore$ Factor of safety against crushing $= \frac{10}{2{\cdot}8945} = 3{\cdot}45$

Now the factor of safety against sliding $= \mu W/P$

Where μ is the coefficient of friction.

$\therefore$ Factor of safety against sliding

$$= \frac{0{\cdot}667 \times 216120}{112500} = \mathbf{1{\cdot}28}$$

The defect of this section is that the eccentricity e is very small (3·51 ft.) though it can be increased up to $b/6$ or 8 ft. in length. That is, the section is somewhat massive and also the factor of safety against crushing over the allowed safe stress is also large. Therefore the base width of the section can be reduced and hence the section is uneconomical.

Problem No. 8.

Adopt a suitable section for a solid gravity dam with the following data:

(1) Height of dam 252 ft.
(2) Maximum permissible inclined stress in masonry 25·2 tons/ft.^{2}
(3) Weight of masonry 160 lbs./c. ft.
(4) Full uplift at heel with uplift area factor of 0·6
(5) No drainage — No tail water
(6) Allowable Coefficient of friction in joints and bed is 0·75

Work out the factor of safety against sliding and the eccentricity of the resultant when the reservoir is full.

Solution: The specific gravity of the masonry is given by

$$\frac{160}{62 \cdot 6} = 2 \cdot 56.$$

The crest width of the dam is given by the equation

$$a = \sqrt{H},$$

where a is the crest width and H is the height of the dam.

$$\therefore \quad a = \sqrt{252} = 16 \text{ ft. (nearly).}$$

Let the section of the dam be rectangular up to a depth of x ft. from top, and then let the downstream face have a suitable batter.

The depth x is given by the equation $x = a\sqrt{\rho}$

$$= 16\sqrt{2 \cdot 56} = 25 \cdot 6 \text{ ft.}$$

Now let the base width of the dam be 'b' and let the resultant act at a distance of $2b/3$ along the base at O from the vertical upstream face for the safety of the dam.

Now referring to Fig. 141.

(i) Horizontal water pressure

$$P = \frac{62 \cdot 5 \times 252^2}{2} = 1984500 \text{ lbs. wt.}$$

$$\text{Its moment about } 0 = -\left\{1984500 \times \frac{252}{3}\right\} \text{ ft. lbs.}$$

(ii) Weight of portion

ABCFE $= W_1 = 16 \times 252 \times 160 = 645120$ lbs. wt.

Its moment about $0 = 645120 \times \frac{2}{3}(b - 12)$ ft. lbs.

(iii) Weight of portion

$$\text{DEF} = W_2 = \tfrac{1}{2} \times (252 - 25\cdot6) \times (b - 16) \times 160$$
$$= 18112\,(b - 16) \text{ lbs. wt.}$$

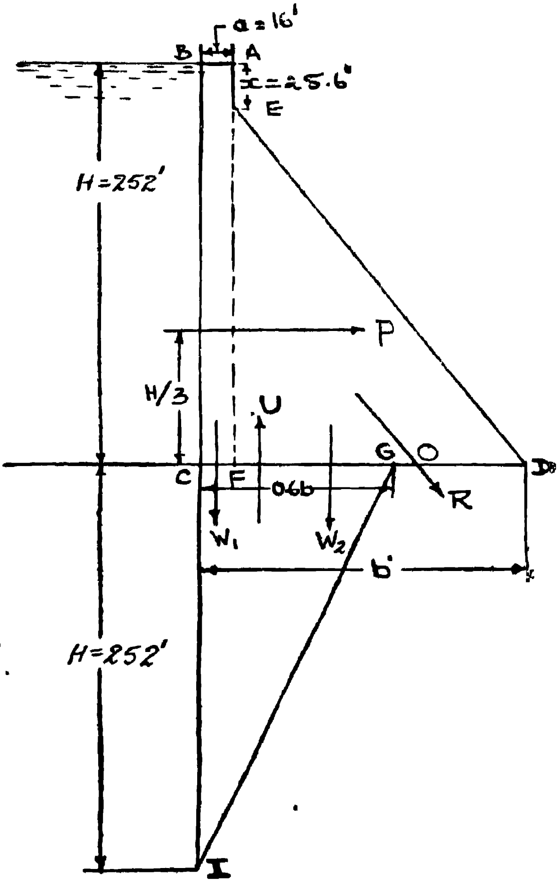


Fig. 141.

$$\text{Its moment about } 0 = 18112\,(b-16) \times \frac{(b-32)}{3} \text{ ft. lbs.}$$

(iv) Uplift pressure $U = \tfrac{1}{2} \times 252 \times 0\cdot6\,b \times 62\cdot5$
$$= 4725\,b \text{ lbs. wt.}$$

Its moment about $0 = -\left\{4725\,b \times \frac{1{\cdot}4\,b}{3}\right\}$ ft. lbs.

$\therefore$ Sum of the moments of all the forces about 0

$$= 645120 \times \tfrac{2}{3}(b-12) + 18112\,(b-16)$$

$$\times \frac{(b-32)}{3} - 1984500 \times \frac{252}{3} - 4275\,b \times \frac{1{\cdot}4\,b}{3}$$

This should be equal to the moment of the resultant R about 0, i. e. they should be equal to zero.

$$\therefore \quad 1984500 \times \frac{252}{3} = 645120 \times \frac{2}{3}(b-12)$$

$$+ 18112\,(b-16)\,\frac{(b-32)}{3} - 4725\,b \times \frac{1{\cdot}4\,b}{3}$$

$\therefore \quad b^2 + 36{\cdot}61\,b - 44038 = 0$

$\therefore \quad b =$ **192** ft. (nearly).

Substituting this value of b we get

$W_2 = 11812\,(192 - 16) = 3187712$ lbs. wt.

$U \;= 4725 \times 192 = 907200$ lbs. wt.

Now the resultant downward force $= W_1 + W_2 - U$

$= 645120 + 3187712 - 907200 = 2925632$ lbs. wt.

The resultant of the downward force and the horizontal water pressure is given by $R^2 = P^2 + W^2$,

where P is the horizontal water pressure

W is the resultant downward pressure

and R is the resultant of P and W

$\therefore \quad R^2 = 1984500^2 + 2925632^2 = 137257525 \times 10^5$

The Inclined stress in masonry $= \dfrac{R}{b \cos\theta} = \dfrac{R}{b{\cdot}W/R} = \dfrac{R^2}{bW}$

$$= \frac{137257525 \times 10^5}{192 \times 2925632 \times 2240} = 11 \text{ tons per sq. ft.}$$

This inclined stress is less than the allowable stress and hence the design is safe.

Factor of safety against sliding

$$= \frac{\mu W}{P} = \frac{0{\cdot}75 \times 2925632}{1984500} = \mathbf{1{\cdot}13.}$$

Eccentricity when reservoir full is given by

$$e = \frac{2b}{3} - \frac{b}{2} = \frac{b}{6}$$

$$= \frac{192}{6} = \mathbf{32\ ft.}$$

Problem No. 9.

Design of a high dam: The book "Principles of Water Works Engineering" by Messrs. Tudsbery and Brightmore gives the detailed design for a dam of height 150 ft., and the same is reproduced here to elucidate the method.

Three fundamental formulæ are employed in this method and they are:

Formula I. $u = \frac{b}{3}\left(2 - \frac{s}{2S_1}\right)$

where u is the distance of the centre of pressure from that end of the horizontal base at which the stress is the greatest,

b is the length of the joint or the width of the base,

S_1 is the mean stress.

Formula II. $b = \sqrt{\frac{wH^2}{\lambda}\left(1 + \frac{w^2H^4}{4W^2}\right)}$

where b is the width of the base

w is the weight per c. ft. of water in tons

H is the depth of water in feet

λ is the limiting stress in the masonry, in tons per sq. ft. and W is the vertical force due to the weight of masonry and of any water lying over any sloping section of the upstream face.

Formula III. $\frac{\rho w d_1}{24}\left\{3b_0^2 - b_1^2 + 6x_1(b_0 + b_1) + 2b_0 b_1\right\}$

$$- \frac{wx_1}{12}(H + H_1)(2b_1 - 3x_1) \doteq W_0$$

$$\ldots \times \left(\frac{b_1 - b_0}{3} - x_1\right) = 0$$

where ρ is the specific gravity of masonry of the dam

d_1 is the vertical depth of the new strip added below the base b_0

b_1 is the base of the lamina below b_0

x_1 is the projection of the end of the base b_1 above base b_0 i. e. upstream of A′ (Vide Fig. 142).

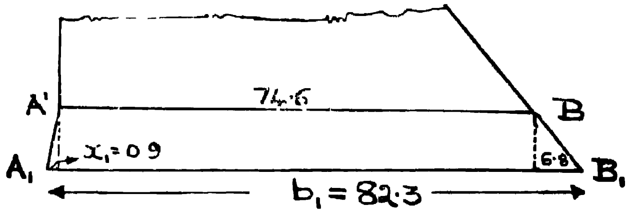


Fig. 142.

b_0 is the width of the base at the bottom of the low dam.

w is the weight per c. ft. of water

W_0 = weight of masonry above the base + vertical component of water pressure, and

H_1 is the depth of water at base b_1

The top portion of 111 ft. is designed as usual, like a low dam, and the bottom portion, i. e. below 111 ft. up to 150 ft., will have to be designed separately, as detailed below:—

Design of the top portion or lower dam portion: This has already been explained previously. Assuming the top width of the dam to be 10 ft. and $\rho = 2{\cdot}25$ and with a limiting pressure of 10 tons, the depth of the dam is given by the formula

$$\lambda = w\,(\rho + 1)\,H$$

$$= w\,(2{\cdot}25 + 1) \times H$$

$$\therefore \quad H = \frac{10}{1/36 \times 3{\cdot}25} = \frac{10 \times 36}{3{\cdot}25} = \mathbf{111\ feet.}$$

The base width at the depth of 111 ft.

$$b_0 = \frac{111}{\sqrt{e}} + \frac{10}{16} = \mathbf{74{\cdot}6\ feet.} \quad \text{(See Fig. 143)}$$

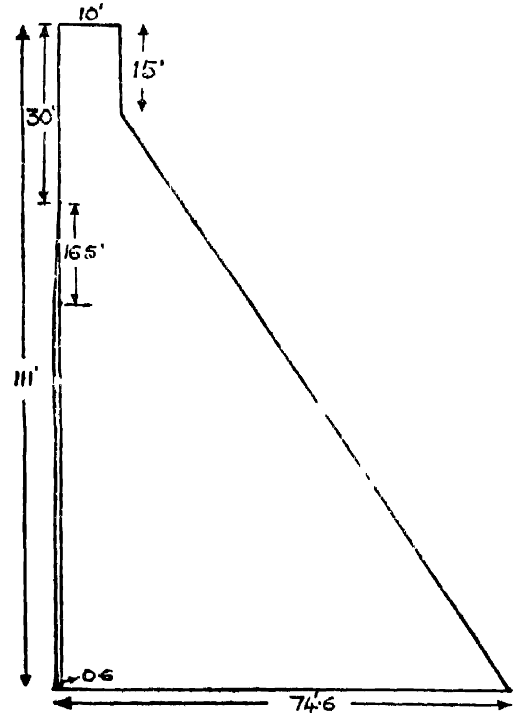
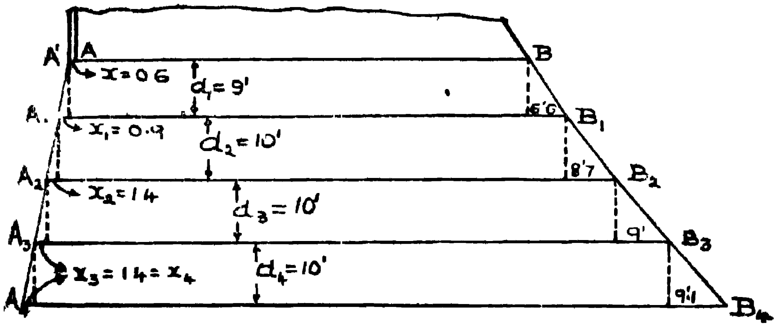


Fig. 143.

Design of the lower portion of the dam: For this purpose, the vertical space from 111 to 150 feet depth will be divided into 4 horizontal laminae (numbered 1, 2, 3 and 4 with base of length b_1, b_2, b_3 and b_4) situated below water at levels

$$H_1 = 120 \text{ ft. } H_2 = 130 \text{ ft.; } H_3 = 140 \text{ ft.; } H_4 = 150 \text{ ft.}$$

Correspondingly, the vertical depth of each lamina is marked d_1, d_2, d_3 and d_4 and the projections of each base upstream of the one above it, x_1, x_2, x_3 and x_4. (See Fig. 144).



HIGH DAM

Length of AB = 74
and A'B = 74 6
A_1B_1 = 82·5
A_2B_2 = 92·5
A_3B_3 = 102·8
A_4B_4 = 113·3

Fig. 144.

The vertical component of the weight of the masonry and water above each base, is taken as W_1, W_2, W_3 and W_4.

First Lamina: $W_0 = 265$ tons, $H = 111$ fl., $b_0 = 74{\cdot}6$ leet and $x = 0{\cdot}6'$.

Approximate weight of lamina from 111 to 120 below water

$$= \frac{9 \times 74{\cdot}6 \left(1 + \frac{120}{111}\right)}{2 \times 16} = 44 \text{ Tons.}$$

Then as a first approximation, $W_1 = 265 + 44 = 309$ Tons.

Substituting these values in Formula II,

$$b_1 = \sqrt{\frac{1 \times (120)^3}{36 \times 10}\left(1 + \frac{1^2 \times (120)^4}{4 \times (309)^2 \times (36)^2}\right)}$$

$= 82{\cdot}5$ ft.

Now, from formula III, the value of x_1 is to be found

$$\frac{9}{16 \times 24}\,[3 \times (74{\cdot}6)^2 - (82{\cdot}5)^2 + 6 \times 157{\cdot}1 \times x_1 + 2 \times 74{\cdot}6 \times 82{\cdot}5]$$

$$-\frac{x_1}{36\times 12}\left[231(165-3x)-265\left(\frac{7\cdot9}{3}-x_1\right)\right]=0$$

$$=1\cdot6x_1^2+199x_1-177=0 \qquad x_1=\frac{3}{3\cdot2}\text{ or }0\cdot9\text{ ft.}$$

From the approximate values of b_1 and x_1, (thus found,) make a second calculation of the weight of the lamina plus that of the water on its upstream face, (44·2 + 2·8), and W_1 will be found to be 265 + 47 = 312 tons.

Substitutions of this value in formula II gives a revised value $b_1 = 82\cdot3$. The difference is too small to affect the value of x_1, which thus remains 0·9 ft.

The results are therefore, $W_1 = 312$ tons: $b_1 = 82\cdot3$ ft.; $x_1 = 0\cdot9$ ft. Therefore $x = x_0 + x_1 = 0\cdot6 + 0\cdot9 = 1\cdot5$ ft.

Second Lamina : Here H_2 is 130 ft.

The weight of the lamina produced from the profile above (first approximation + overlying column of water)

$$=\frac{10}{16}\left\{82\cdot3+\frac{10}{9}-\frac{1}{2}(82\cdot3-74\cdot6)\right\}+\frac{120+130}{2\times36}\times1\cdot0=58\text{ tons}$$

W_2 (first approximation) = 312 + 58 = 370 tons.

Applying this to Formula II, we get

$$=\sqrt{\frac{(130)^3}{360}}\left(1+\frac{(130)^4}{5184\times(370)^2}\right)=92\cdot5\text{ ft.}$$

Applying the value of b_2 as got above in formula III we get

$$x_2 = 1\cdot4\text{ ft.}$$

From these first approximations of b_2 and x_2, recompute the weight of the second lamina + water

$W_2 = 312 + 59 = 371$ tons.

Recalculation of formula II with the above value of W_2 gives $b_2 = 92\cdot4$ ft.

The values at the base of the second lamina are therefore

$W_2 = 371$ tons; $b_2 = 92\cdot4$; $x_2 = 1\cdot4$

Therefore $x = 0\cdot6 + 0\cdot9 + 1\cdot4 =$ **2·9 feet.**

Third Lamina : $W_3 = 371 + 67 = 438$ tons.

$b_3 = 102\cdot8$ feet and $x_3 = 1\cdot4'$.

These are as a first approximation. The assumed values from prolonging the slopes of the trapezoid above, are so nearly correct that no second approximation need be worked out.

For the third lamina the base dimensions are

$W_3 = 438$ tons; $b_3 = 102{\cdot}8$ ft.; $x_3 = 1{\cdot}4$ ft.

Therefore $x = 2{\cdot}9 + 1{\cdot}4 = \mathbf{4{\cdot}3}$ **ft.**

Fourth Lamina. Similarly the fourth lamina gives

$W_4 = 513$ tons; $b_4 = 113{\cdot}3$ ft.; $x_4 = 1{\cdot}4$ ft.

Therefore $x = 4{\cdot}3 + 1{\cdot}4 = \mathbf{5{\cdot}7}$ **ft.**

The lower lamina fulfil the following conditions for "reservoir full".

(a) Weight of masonry + water lying on the upstream face "reservoir full" acts at the upstream extremity of the centre third.

(b) Maximum stress is limited to 10 tons per sq. ft.

It does not however necessarily follow that the maximum stress, "reservoir empty," might not exceed 10 tons. This should be examined by the method of moments, separately.

Problem No. 10.

An overflow masonry dam is 100 ft. high. Its upstream face is vertical, and the depth of water over the crest is 10 ft. It has a velocity of approach of 4·9 ft. per second. Determine the profile of the downstream face of the dam.

Solution:—The total head = Depth over crest + Depth due to the Velocity of approach.

$$\text{Head} = 10 + \frac{4{\cdot}9^2}{2g} = 10 + 0{\cdot}375 = 10{\cdot}375$$

$$\text{Again, } x^2 = \frac{c^2hy}{7{\cdot}67}.$$

This is got by assuming Bazin's theory that the thickness of water running over the crest is 0·69 of h.

c is taken as 3·98, and $h = 10{\cdot}375$.

$$\text{Therefore, } x^2 = \frac{3{\cdot}98^2 \times 10{\cdot}575 \times y}{7{\cdot}67} = \frac{164{\cdot}3}{7{\cdot}67}y$$

$$\text{or } x = \sqrt{22{\cdot}3y}$$

The profile of the rear face may be determined, by giving different values to y in the above equation, as noted below: (See Fig. 145).

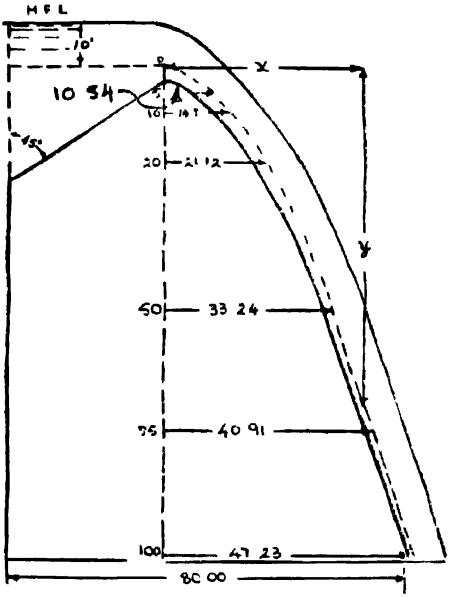


Fig 145

y	0	5	10	20	50	75	100
x	0	10·54	14·94	21·12	33·24	40·91	47·23

Questions

Distinguish between : Core wall and Cut-off.
(Gujerat University, April 1955)

Write short note on : Stresses in Gravity Dam.
(Gujerat University, April 1955)

Distinguish between the theoretical and practical profile of a masonry dam. Draw up a tabular statement of the design data

needed for a cement concrete dam giving the usual or assumed values and state very briefly the design procedure.

(Poona University, April 1955)

Explain the function of the following, indicating the difference between them : U/s blanket and Cut-off.

(Poona University, April 1955)

Describe in detail the important points involved in the modern method of constructing a cement concrete dam.

What are the advantages of a cement concrete dam as compared with a masonry dam ?

(Poona Uni. April 54)

(1) Write short notes on the following : (a) Expansion joints in dams. (b) Uplift in dams.

(A. M. I. E., May 1951)

(2) Describe four types of masonry dams, and discuss the merits and demerits of each.

(A. M. I. E., November 52)

(3) Design a masonry dam with the following data :—

Height from foundation to H. F. L. ...	150 ft.
Safe stress in masonry	18 tons/sq. ft.
Top width	12 ft.
Weight of masonry	150 lbs. per c. ft.
Free board	6 ft.

What is uplift in a dam ? What is its effect ? What is its amount ? How can it be reduced ?

(Bombay Univ. April 49, B. E.)

(4) Write a short note on : Grouting of foundation of a dam.

(Bombay Univ. Oct. 49, B. E.)

(5) What are the forces to be taken into account in the design of an Overflow type gravity dam ? In what respect do they differ from those of a Non-overflow type gravity dam ? Deduce an expression to determine the shape of crest of an overflow type dam to obtain maximum hydraulic efficiency.

(Poona University, November 1953 B. E.)

(6) Write short notes on the following :—(a) Low dam and high dam. (b) Drainage gallery in Masonry dams.

(Guj. Univ. April 53, B. E. and S. E.)

(7) What is uplift in the case of a dam? Explain how it affects masonry and earth dams respectively. Describe the measures adopted during construction to reduce uplift. Can uplift be dealt with in an existing masonry dam?

(Gujerat Univ. April 1955)

(8) What is meant by uplift? What is its importance and how can it be reduced in the case of dams? What would be the value of the coefficient of uplift in the case of a structure founded on alluvial soil?

(Dept. of Technical Education, Bombay State, April 1954)

(9) What are the principles involved in the design of a gravity dam? What is meant by the 'Elementary Profile' of a gravity dam and how is it deduced?

(Mysore University, March 1947)

(10) Write a short note on: Drainage Gallery.

(Mysore Univ. March 48)

(11) How is a masonry dam designed against "Uplift" and "Flotation"?

(Mysore Univ. March 1948)

(12) Explain briefly the principles involved in the design of masonry gravity dam of about 200 feet height. If you are asked to choose between surki and cement mortar for its construction which would you prefer? Give reasons. Explain the use of drainage gallery in a High dam.

(Mysore Univ. April 49),

(13) Mention the several types of masonry dams that are usually constructed. Indicate the situation in which each is best suited.

(Mysore Univ. March 1950)

(14) Enumerate the principles of design of masonry dams. Deduce an expression for the compressive stress induced in a masonry dam, and calculate the limiting height for a low dam of rubble masonry in surki mortar, assuming suitable values for the safe compressive stress and specific gravity.

(Mysore Univ. September 1950)

(15) Distinguish between a high and low dam. Explain the principles on which high dams are designed. Discuss the

merits and demerits of using surki mortar in the construction of masonry dams.

(Mysore Univ. March 1951)

16. Distinguish between overflow and non-overflow gravity dams. Explain the requirements of stability of each.

Derive an expression for the design of down stream profile of an overflow dam.

Design the profile of a gravity concrete dam for the following data. Free board is 6 ft.

(Bom. Univ. B. E. Civil (old.) April 1954)

17. Find the factor of safety at the base of a gravity dam against failure due to (a) crushing and (b) sliding with the following data.

Maximum depth of water	60 ft.
Free board	4 ft.

Section of dam is rectangular 12 ft. wide at top for a height of 16 ft. thereafter the U/S face is vertical and the D/S face has a batter of 1:0·75 (vertical:horizontal). Weight of masonry is 160 lbs./c. ft. Assume full uplift pressure over half the area. Maximum allowable compressive stress in masonry is 10 tons/sq. ft. coefficient of friction is 0·667.

(Bom. Univ. B. E. Civil, (new) Nov. 1954)

18. What are the requirements of stability of a masonry (gravity) dam ?

For computing stability of a gravity dam the cross-section can be divided by several horizontal lines into 'Zones' according to certain limiting conditions. Explain this statement by showing nature and extent of these zones in the case of (i) A non-overflow type dam, and (ii) An overflow type dam.

(Bom. Univ. B. E. Civil. (new) April 1955)

19. Write short notes on Overflow dams.

Explain clearly the difference between Anicut and dam.

(Mys. Univ. B. E. Civil. Sept. 1955)

20. List the forces involved in the design of a masonry gravity dam and give their magnitudes and points of application. What are the essential conditions that a gravity dam should satisfy for its stability.

State under what conditions would you prefer an earthen dam for a gravity dam.

(Bom. Univ. B. E. Civil (New) Oct. 1955)

21. Write explanatory notes on : (a) Methods of preventing cracks in mass concrete, and (b) Trial-load twist method of analysing a gravity dam.

What is meant by Limiting Height of a Low-dam ? Work out its value making necessary assumptions.

Enumerate different methods of designing a High (gravity) dam and explain one of those methods in detail.

(Bom. Univ. B. E. Civil Oct. 1956)

22. Write short notes on Drainage gallery.

(Mysore Univ., B. E. Civil., April 1957)

(a) What is uplift on a masonry dam ? What causes this uplift ? What is its effect ? How can it be reduced ?

(b) Design a gravity dam of stone masonry having a specific gravity of $2\frac{1}{4}$, to store 90 ft of water. Sketch its cross section and dimension the parts, in accordance with usual practice. Explain how this section is checked for its stability graphically.

(Mys. Uni., B. E. Civil, Sept. 1954)

23. Distinguish between a ' High and Low ' dam. Discuss briefly the conditions of stability of a gravity dam. Explain the use of drainage gallery provided in modern dams.

Deduce an expression for the elementary profile of a masonry dam.

Design a gravity dam of stone masonry of specific gravity $2\frac{1}{4}$, to 110 feet of water. Sketch its cross section and dimension the parts in accordance with usual practice.

(Mys. Uni., B. E. Civil, Sept. 1956)

24. What are the important conditions for the stability of a gravity dam ? Determine the downstream slope for a theoretical profile in terms of base width b, height h, upstream water pressure, uplift coefficient c and the specific gravity of the materials.

(Gujarat Univ. April 1957)

Write short notes on : Free roller gates.

(Gujarat Univ. April 1957)

CHAPTER XI

ARCHED AND BUTTRESS DAMS

1. Arch Dams: An arched dam is an ideal dam containing much less material than the other types. It is well adopted where conditions permit. The weight of the masonry is not taken into account in resisting exterior loads. So an arched dam easily resists any uplift on the base.

An arched dam may be defined as a curved dam which carries the major portion of the load on it (water load) by arch action, horizontally to the abutments; and the section of the dam adopted is dependent on the curvature.

2. Classification: Arched dams may be divided into three types as noted below:

(i) Constant radius arch dam,
(ii) Constant angle arch dam,
(iii) Arch dam with variable radius.

(i) *Constant Radius arch dam*: Here the radius of the horizontal arch ring at any level is constant, but the angle subtended varies.

A constant radius arch dam contains a vertical upstream face. Vertical batters are provided near the base of higher cross section, though the dam may be high having extrados curves whose radii gradually increase towards the lower part of the gorge. When compared to the extrados curves, the intrados curves may be concentric. To overcome high reservoir pressures, and thus to increase their thickness, their radii usually decrease as the depth below the crest of the dam increases. Constant radius arch dams are generally adopted for U shaped gorges, were at lower elevation, a relatively heavier water load is carried by cantilever action. This type is not much recommended.

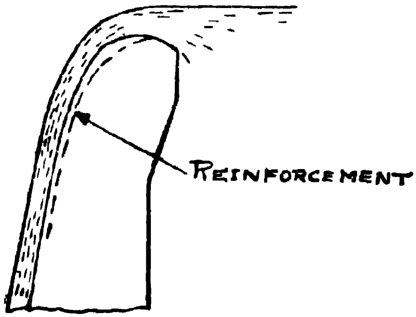
(ii) *Constant angle arch dam*: Here the central angle is constant and is roughly kept about 120 to 150 degrees. It is seen that the constant angle arch dams are more economical, and hence this type is more used than the constant radius type.

As the depth below the crest of the dam increases, these dams generally have extrados curves and intrados curves with decreasing radii, thus keeping the central angles of the arch as large as possible. This central angle, for all practical purposes, is generally taken to be 120 degrees. This large central angle enables maximum arch efficiency to be obtained at all elevations.

This type can be built with about ¾ of the quantity of masonry required for a constant radius arch dam. The construction produces an overhang. The lower arches undercut the upper ones on the upstream side. This hampers the construction. Hence this type is an ideal which cannot generally be fully attained.

This does not apply where there is heavy flood discharge, because it allows a vertical downstream face which damages the foundation. As the downstream face is to be shaped to fit. in with

OVERHANG OF THE LIP OF CREST



Of the dam, to throw off the sheet of water

Fig. 146

the sheet of falling water, in this case, a vertical upstream face and with varying radius, will keep the arch stress within the limit. Also the lip of the crest may be made to overhang, in order to reduce the throw of the sheet of water. (See Fig. 146).

(iii) *Arch dam with variable radius* This type is only a compromise between the types (i) and (ii) above. Neither the radius nor the central angle is constant in this type. Even here, the value of masonry is saved to a large extent (it can be) built with about 80% of the masonry required for the constant radius

type). This type is widely used, adopting either the cylinder or the elastic theory (as will be explained later in the Chapter).

3. Site: Arch dams are well adapted to canyons or gorges where the shape of the gorge is either U-shaped or V-shaped, the former being suitable for constant radius dam, and the latter for constant angle arch dam. A single arch dam is very economical when the site selected is deep and narrow.

4. Construction: Arch dams were first being constructed of rubble, ashlar and cyclopean masonry, but recently owing to the popularity and availability of cement, these dams are constructed mostly of cement concrete.

Since the effect of temperature is easily noticeable, special precautions are to be taken in the construction of the arch portion, especially (to protect) the lower portion. In this portion, vertical cracks are apt to occur, if there is an appreciable decrease in temperature, in which case the arch becomes useless.

5. Principles of Construction: Arch dams are built on one of the following theories.

(i) The cylinder theory,

(ii) The elastic theory,

(iii) The trial load analysis method.

(1) Cylinder Theory: It is assumed here, that the arch ring acts like the shell of a vertical thin cylinder and it rests freely on the abutments, and the whole pressure is transferred to the abutments.

The principles on which this theory is based are :—

(a) The arch dam sustains, by arch action, the water pressure on it.

(b) The pressure acts normally on the surface in contact, and radial to the arch.

(c) The line of pressure through the arch corresponds to its curvature.

(d) The pressure is transferred to the abutments through each horizontal arch ring. (The abutments are the sides of the gorge which is spanned by the arch).

The cylinder theory is applicable to simple concentric circular rings subject to uniform radial stresses. This methed is not very

accurate, because an arch dam is never a complete cylinder, and as such the stress and the dimensions computed are only approximate, and hence it is used only for preliminary designs and estimates or for small dams in simple settings.

N. B. The Cylinder Theory is not correct if the radius divided by thickness of arch is very small.

(2) Elastic Theory : An arch slice of a dam (arch dam) is not a complete ring by itself, and as such the cylinder theory as described above, is not scientifically correct and the result should only be taken as approximate.

Deformity (Elastic) :—Owing to external load or water pressure (in the case of storage dam) the arch ring shortens uniformly throughout, without any change of shape (as the load is uniform). That means to say, the arch length is shortened and the span remains unchanged. Thus the dam is deformed and fresh stresses *(rib shortening stresses)* are produced especially with thick arched dams and small angles. These play an important part.

Varying load :—In an arched dam, the loading is irregular and varying. Thus there is elasticity produced in the arch, and hence the elastic theory is appropriate. This varying load may be due, not only to the irregular loading as above, but also due to the inclined or irregularly shaped arches and variable silt pressure, and sometimes due to earthquake.

Temperture and plastic flow of concrete :—When there is a drop in the temperature, there is shrinkage, especially when the construction is of concrete; or there is shrinkage when concrete dries. Also, when there is a rise in temperature, there is a slight expansion. These causes lead to the adoption of the elastic theory.

Abutments yielding :—The abutments on which the arch rings rest are also elastic. They spread apart slightly by the thrust of the arch. Under these circumstances, the elastic theory is more recommended than the cylinder theory.

(3) Trial Load Analysis. The cylinder theory and the elastic theory will only give a rough idea about the stresses. In these two theories, the elements concerned are taken independently of each other. They are taken as separate arches whereas actually the adjacent arches restrain each other and the stresses in them are complicated. A correct mathematical analysis or calculation

is not practicable and therefore a method known as Trial Load Method has recently (1922-23) come into use, and is being adopted.

This method takes into account all factors affecting it. In this method, the dam is assumed to consist of two systems (i) horizontal arches and (ii) vertical cantilevers. Each of these systems occupies the whole and hence the loading is to be divided between them, so that the deflections in both are identical. This division is done in such a manner, that the deflections are coincident at all points. This is done by a series of trials.

As trials are to be made first, there should be a ready made design for getting on with the work. This is done by any of the above two methods, either the cylinder theory or the elastic theory. As stated above, the dam structure is divided into horizontal elements comprising a series of arches, one unit distance apart, and also vertical elements one unit apart. The two elements, vertical and horizontal, intersect in a common prism. A portion of the load on the face of the prism will be borne by the arch and the remainder by the cantilever.

The loadings are (i) primary load, and (ii) secondary load. The primary load is due to the horizontal water pressure and is divided between the vertical cantilevers which take the load to the foundation and the arch rings which take the load to the abutments, in such a manner that the radial deflections of the secondary load and part of the primary load are equally distributed on both the cantilever and the arch ring. The resulting arch stress is thus calculated and the thickness of the arch found out. How much (water) load each takes is to be arrived at only by trial and error, hence the name *trial load method*.

(For an arch dam, roughly 10 divisions, of both cantilever and arch ring, are usually made for purposes of calculation).

Proper balancing of the elements gives the results reasonably correct. The important influences are radial deflection, and radial load. These elements must be brought to approximate arrangement, before introducting any tangential shear or twist.

The first trial division may not be accurate, but with a moderate number of trials, close approximation could be got. Typical load and deflection from trial analysis of symmetrical arch of a simple cross section are plotted. Radial, tangential and twist data are shown for 3 out of 9 cantilevers. Other forces like

earthquake, inertia of concrete, increase of water pressure, are shown on the water pressure force diagram.

This is the most accurate method available and widely used nowadays, but it is complex and tedious.

In this method, special precautions should be taken with regard to the following points.

(1) The effect of foundations and abutment developments should be considered.

(2) Proper distribution of loads will have to be determined by equating the calculated deflexions at each conjugate point, from different systems of load transference.

(3) In addition to the ordinary arch and cantilever actions part of the load is carried by tangential shear.

6. Comparison of the 3 Theories

Cylinder Theory	Elastic Theory	Trial Load Analysis
1. Used for the design of simple slender arches.	Used for structures of all shapes and all loadings.	Used for important structures, of unusual profile conditions where a more complete knowledge of stress distribution is desired.
2. Applied only to a simple concentric circular ring, subject to a uniform radial loading.	There is no restriction either to the shape of the structure or to the loading. Forces due to earthquake, and variable silt-pressure are taken into consideration.	Applies to separate arches, each acting independently.
3. For thick arches, it is a poor indication of stress (if there are special loadings, and also variation in temperature).	This type gives a better idea of actual stresses, and permits allowance for variations in temperature, foundation yielding, and irregular arch forms, etc.	

7. Design: On account of the uncertainty as to the actual stresses in an arch dam, the design made should be compared carefully with existing structures similarly situated, and proper allowance made for all variations (if any) before the final design is made. Also due to the above uncertainties, a higher factor of safety (say even 8 to 12) should be adopted.

In very high dams, the arch stresses near the base are reduced considerably, to provide for additional width, so that the pressure on the foundation may be properly distributed within the safe limits, also the thickness is intended more at the base, to limit the stresses due to vertical beam action.

The greatest economy in arch design is got, when the central angle is made 133°–34′.

When the reservoir is empty, the weight over the masonry should not exceed the maximum stress. While calculating the stability of the arch dam, only water pressure is taken into the consideration, and not the weight of the masonry.

(a) *Stress Under Water Pressure:*—From cylinder theory, thrust on a horizontal ring is R × P, where R is the radius to the centre of arch ring, and P is the water pressure per sq. ft. If S_m be the mean stress in a horizontal ring, then

$$S_m = \frac{RHw}{b}$$

where H is the depth of water,

w is the weight of water per unit, and

b is the width of arch ring.

Therefore, we have $b = \dfrac{RHw}{S_m}$.

N. B.—The larger R is made the greater b becomes, hence it is advisable for an economic section to have R small. That is, the gorge should be narrow, and if b is made too small there is a likelihood of percolation. If b is very small, when compared to R, then S and S_m become nearly equal where S is the maximum stress. If not, the relationship between the mean and maximum stresses is arrived at by the equation,

ARCHED DAM

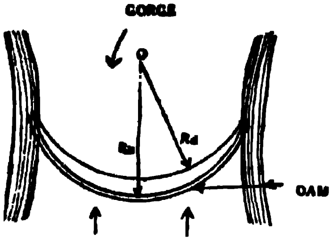


Fig. 147.

$$S = \frac{S_m \cdot 2R_u}{R_u + R_d}$$

where R_u is the radius of the upstream arch ring, and R_d is the radius of the downstream arch ring. See Fig. 147.

Therefore,

$$2\,R_u$$

and $\quad R_d = R_u - b.$

If the limiting stress or maximum stress is denoted by λ, then,

$$S \quad \lambda = \frac{2\,HR_u^2\,w}{2b\,R_u - b^2}$$

$$= \frac{2\,H\,w}{\frac{b}{R_u}\left(2 - \frac{b}{R_u}\right)}$$

$$\text{Therefore, } b = R_u\left(1 - \sqrt{1 - \frac{2Hw}{\lambda}}\right)$$

(b) *Profile* :—The theoretical profile of an arch dam is a triangle, just like the one for a gravity dam. Hence for all practical purposes a crest width is essential. This width is generally kept smaller than that of a gravity dam, the minimum being $3\frac{1}{2}$ ft.

Bligh has given the width to be $\frac{\sqrt{H}}{2}$.

(c) *Maximum Stress* :—An arch dam generally takes a higher stress than an ordinary gravity-dam. For dams used in Australia, it is seen that these stresses vary from 15 to 20 tons per sq. ft.

(d) *Slenderness Ratio* :—This is the ratio between the curved length of the arched dam and the thickness of the arch.

This ratio should not exceed

75 at the top and

25 at midheight

Slenderness ratio of some dams is given below :

North Crow Dam at Cheyenne, Wyo.		Las Vegas New Mexico	Practically
At top	35	42	60
At midheight	11	17	20

The ratio at the top may be somewhat increased, if the thickness increases rapidly towards the base and the ratio at midheight is proportionately reduced. This is done generally when a vertical reinforcement is used.

8. Examples : (a) *Boulder Dam :* During the construction of this dam, the main obscure facts of the trial load method were first substantiated here, and the method received its greatest advancement.

It is the most notable dam in the world, the height of the dam being 726·4 ft. above the foundation. The base width is 660 ft. It is a gravity type arch dam, being curved in plan.

It was one of the first dams to be constructed with low heat cement and the first in which artificial cooling methods were used on a large scale. The structure was divided both axially and transversely by contraction joints (1) to control cracks due to shrinkage and (2) to ensure adequate arch action ; and the result of this was a columnar effect. After cooling, the joints were grouted to become a monolith.

(b) The Bear Valley Dam :—The construction of an arch dam is very satisfactorily done in the Bear Valley Dam, in California, U. S. A., which is said to be one of the engineering wonders. It was built in 1885, the outer faces of the structure being of uncut granite, and the hearting of rubble. The width at the top is about $2\frac{1}{2}$ to 3 ft., and length of the dam at the top of the gorges is 300 ft. The radius on the upstream side is 342 ft., and the maximum stress realised is about 49 tons per sq. ft. A section of the dam is given in Fig. 148.

SECTION OF BEAR VALLEY Dam (U. S. A.)

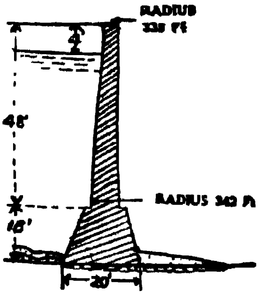


Fig. 148.

BUTTRESS DAMS

(1) Classification : Buttress dams may be classified under the following heads :

1 Simple Beam Type
2 Continuous Deck Type
3 Cantilever Deck Type
4 Round Head Buttress Type
5 Multiple Arch Type

(1) *Simple Beam Type*: This is the type most commonly used.

As an example of this, the Rodriguez dam (in Mexico) may be taken. The height of this dam is 240 ft. The slab has a thickness of 2′-1″ at top and 5′-6″ at bottom, with an upstream slope of 45 degrees. The buttresses are spaced 22 ft. apart, with a thickness of 1 ft. 7 in. at top and 5 ft. 6 in, at botton. The width of the footing is 365 ft.

(2) *Continuous Deck Type*: If there is any unequal settlement of the foundation, the action of the continuous deck is uncertain. Joints are required here and there, to reduce shrinkage and temperature effects.

(3) *Cantilever Deck Type*: This is not generally in use.

(4) *Round Head Buttress Type*: The buttress heads are constructed to the full width of the span between the buttresses. The water pressure is made to act directly, as a compression stress on the buttress.

(5) *Multiple Arch Type*: Here the water load is taken by a series of inclined arches which span between the buttresses. This type is stronger than the simple beam type. The slabs may be made thinner. The buttresses may be spaced further apart. But they should be made thicker and more substantial. In some cases, double-walled or hollow buttresses are used. When there is tension reinforcement is used,

For this type, *the trial load analysis* is not required. The arch faces are treated as simple *cylindrical* faces.

As an example of the Multiple Arch, the Barlett Dam may be cited. It is 286 ft. in height, and the upstream slope of the slab is 48 degrees. The thickness of the arch slab is 2 ft. 4 in. at top and 7 ft. at bottom. The angle at the centre is 180 degrees, and the bottom width of the buttress is 360 ft.

The object of the round head is to save any reinforcement, which may be required, if other types are used. This may be taken as a modified gravity dam, free from uplift, and more stable against overturning. It can be used even where the foundations are weak, by providing wide butresses, and if required, spread footings also. By adjusting the upstream face slope, a low sliding factor can be obtained. It can be used safely in earthquake regions also, owing to its flexibility.

(2) Uses : These dams are used where the foundation is not strong, and also for wide rivers and medium height dams. If the dam is long, the arched buttress type is the one best fitted.

(3) Stresses : In the case of an arched buttress dam, the stress in arch portion is considered as in the ordinary single arch dam. For the purpose of calculation of the stress, one pier and half of each of the two arches (one on either side) should be taken into account.

If, instead of arches, the interior is a slab or beam, then the design will have to be worked out, similar to that for a road bridge.

(4) Stability and Design : The section of the dam is inclined at 45° to 55° to the horizontal, and as water rests on the extrados, this makes it stable against overturning or sliding; but due to this water on the slope of the section, there is more weight on it, and some of this goes to the foundation and the remaining to the abutments.

The buttress has to withstand its own weight, in addition to the water load and the load from the arches. All the three act on the foundation.

The section of an arched dam should satisfy all the conditions pertaining to a gravity dam. One unit of the section should satisfy all the conditions of stability, and the stress on the arch should be within the safe limit.

N. B.—One unit means one buttress plus two half arches. See Fig. 149.

The buttresses should be spaced generally 30 to 60 ft. apart, and the central angle for the arch should be 130° to 150° and it should be designed just like a single arch.

For the proper design of the structure, temporary assumptions should first be made and a rough design got. This should be

verified for stability, etc., and when the design satisfies all the conditions required, it should be adopted.

MULTIPLE-ARCH DAM

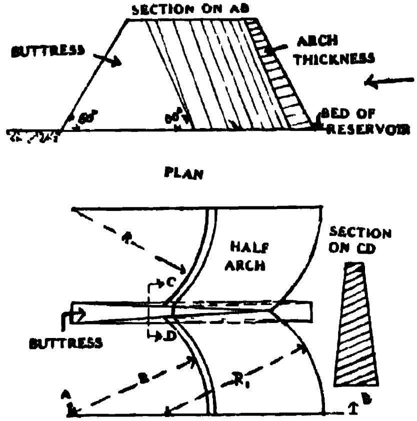


Fig. 149.

N. B.—The downward component of the water pressure is generally greater and the uplift is less. If there is tail-water, full uplift should always be included. If the dam is very high, and the buttress thin in section, these buttresses will have to be designed taking into consideration the wind pressure also, to prevent buckling.

(5) Construction (Arched Buttress): The arches are built usually segmental and inclined at an angle of 30° to 45° to the vertical. The buttresses are triangular, the object being to get a large spread of foundation, so that the multiple arch can be used with safety, even on soft beds of the rivers. This increases the total pressure on the face of the arches, but does not increase the intensity of stress.

The disadvantage in the design is that it has the effect of depressing the intersection of the resultant of the water pressure and the weight of the masonry, thus making the resultant to take a steeper angle with the horizontal, which means increasing frictional resistance to sliding (this should always be guarded against).

The buttress of an arch dam should be constructed at a distance varying from 3/4d to 2d (where d is the depth of water)

depending upon the nature of the foundation. The width of the buttress should be made sufficient for the skewback of the arch to rest on, and the angle at the centre made from 100° to 120°.*

In many cases of multiple arch dams there is usually overflow over the dam, as a separate waste weir is not generally provided. In such cases, in order to prevent damage at the rear toe, an apron or water cushion should be provided for.

(6) Advantages :—(1) This type of dam construction saves 15 to 35 p. c. of cost.

(2) It is better suited to changes of temperature.

(3) Horizontal cracks are less.

(4) The vertical cracks, if prevented, will tend to close.

(5) The foundation need not be so strong as that for a single arch dam.

(6) Buttress foundation can be constructed as for a separate pier.

(7) Where the dam is roughly 100 to 120 ft. high and where the materials are costly, the buttress type dam is economical.

(8) The positions of the line of piers can be varied by varying the slope of the face of the arch.

(9) The weight of water overlying the arch does not increase the stress on the arch ring. Hence any inclination of axis can be adopted,

Disadvantages :—(1) Due to the drawing out or filling of the reservoirs, there is a liklihood of the multiple-arch dam to move under the changing pressure and thus to become weak or even rupture.

(2) There is a chance of leakage of the arched rings through the comparatively thin wall. (This can be prevented by plaster).

(3) As the top thickness is small, future raising of such dams would be difficult. So they are advantageous only for low depths.

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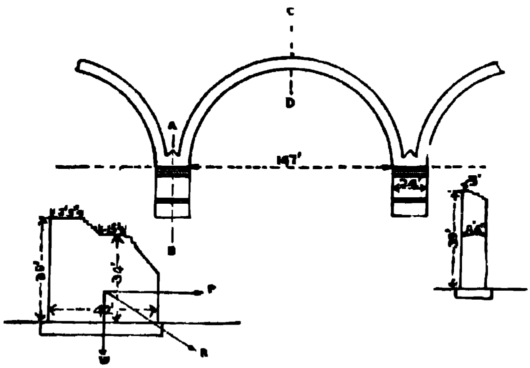
* Theoretically, the most economical arch for a multiple-arch dam must have the central angle equal to 130° 34″ ; but for practical reasons, an angle of 120° is preferred.

(7) Example: An existing example of multiple–arch buttress dam is seen in the Meer Allum reservoir in the (Hyderabad) present Andhra State. This is the first of its kind in India, built in 1809. It is about ½ mile in length, and slightly curved in plan. It consists of 21 arches with required buttresses, and is built on the rocky bed of the river. The span of the arches varies from 70 to 147 ft. and the height of the dam is 40 ft., See Fig. 150.

MULTIPLE ARCH DAM : MEER ALLUM DAM (HYDERABAD) PLAN



SECTION ON AB SECTION ON CD

Fig 150.

(8) Comparison of Gravity & Arch dams.

Gravity Dams *Vs*. Arch Dams:

A dam of the pure gravity type is one, in which the weight of the masonry is the sole factor satisfying the conditions of stability, while a dam of the arched type relies for its stability solely on the arched form it possesses. In it, the pressure of water is transmitted laterally through the horizontal sections in slices to the abutments, i. e. to the sides (of the hill, where they terminate). The thickness of arched dams is so small, that the weight of the masonry falls outside the toe, and hence the dam is in a state of equilibrium, if considered in the way of gravity dams alone without the effects of the arched form. If such a dam fails, it will probably be by the crushing of the masonry at the ends of the horizontal arches. At the present state of knowledge, considering the elastic yielding of masonry, one cannot determine the cause with any considerable degree of accuracy.

Regarding the distribution of pressure over the cross section of the arch: As the arch is supposed not only to transmit the pressure laterally to the abutments at the sides, but as the dam rests on the bottom of the valley also, it is sustained again at these points, so that it cannot act as a true arch.

Hence few dams are constructed purely on this principle, but something of the gravity cross section has also been combined. Such dams (presumably) resist the pressure both by gravity as well as by arch action.

Of the existing type of arched dams the following are good examples,

(1) the Zola dam in France,
(2) the Bear Valley Dam, and
(3) the Sweet Water Dam in America.

9. (a) Comparison of the Different Types of Dams:

	Gravity	Arched	Buttress
1. Nature of Valley	If the valley is narrow and if good rock is available in the foundation and abutments, this type is used.		
2. Waste Weir	This type is used when the discharge is large.		Buttress type is more advantageous when the discharge is large.
3. Climate		If the climate is cold, these two types are not used in order to avoid spalling of concrete.	
4. Roadway	If a roadway is required, this type is well suited.	These are not suited to locate a roadway.	
5. Materials			If materials are not available in good quantity, this type is preferred.

9. (b) Comparison of the different types of dams.

	Gravity Dam I	Arch Dam II	Arched Buttress Dam III
Foundation	Good sound rock is required.	The rocks should be very sound.	Even ordinary rock is sufficient.
Site	River should be broad and shallow.	The river should be in a gorge.	A wide and shallow river is preferred.
Facility for improvement	Not easy to raise the dam, but can be done with some extra cost.	Not practicable.	Not practicable.
Construction	The dam can be built to any height.	This can also be built to the designed height.	Fit only for moderate heights.
Cost	This requires less skilled labour than types 2 and 3 and hence cheaper.	Requires skilled labour. It is more economical than type 1, if radius is small. As a higher stress can be allowed, a thinner section of the dam is possible, and hence less cost.	Requires skilled labour and is economical for only low heights.

For problems refer to the book " Solutions to Problems in Irrigation Engng. " by the authors

Questions

(1) Obtain the base width of an arched dam whose radius to the extrados is R, the maximum permissible compressive stress in masonry is F and the height of water stored h, proceeding on the assumption that the stresses are in accordance with the thin cylinder theory.

(Mysore University, March 1950)

(2) State briefly the principles of design of arch and buttress dams. When are such dams used? What are their advantages?

(Bom. Univ., April 1949, B. E.)

(3) State under what conditions you would prefer an arch dam to a gravity dam. Give reasons.

State the principles of design of an arch dam.

What is Trial Load analysis?

(Bom. Univ., Oct. 1949, B. E.)

(4) Write an explanatory note on: Trial Load analysis.

(Poona Univ., Nov. 1953, B. E.)

(5) What is a single arch dam? When is its construction feasible? Give briefly the principles of design of this type.

(Gujarat Univ., April 1953. B. E.)

(6) Write an explanatory note on: Single Arch Dam.

(Gujarat Univ. April 1953, S. E.)

(7) What is a single arch dam? When is its construction feasible? Explain clearly the modern and accurate method of designing a single arch dam.

(Gujarat University, Nov. 1953, B. E.)

(8) Discuss briefly the principles governing the design of an arch dam.

(Bombay Univ. B. E. Civil (Old Exam.) April 1954)

(9) Discuss in detail the following: Suitable sites for 'Arched Dams' in general and 'Constant Radius' and 'Constant Angle' arched dams in particular.

(Bom. Univ. B. E. Civil (New Exam.) April 1955)

(10) Explain clearly the Cylinder Theory and Elastic Theory of design of arch dams.

(Mys. Univ. B. E. Civil, Sept. 1955)

(11) Write explanatory notes on the Cylinder Theory of the design of arch dams and its limitations in practice.

(Bombay Univ. B. E. Civil. October 1956)

(12) When would you prefer an arched masonry dam to a gravity masonry dam ?

Obtain the base width of an arched masonry dam whose radius to the extrados is R, the maximum permissible compressive stress in masonry is F and the height of stored water is H, on the assumption that the stresses are in accordance with the Thin Cylinder Theory.

(Mysore Univ. B. E. Civil, April 1957)

CHAPTER XII

SLUICES AND WEIRS

(of Tanks)

SLUICES

1. Object :—The object of the outlet is to provide a means for letting out the stored water either for irrigation or for any other purpose.

Every dam should have a sufficient number of sluices to supply water both adequately and conveniently, so that all the lands may be irrigated. Each sluice should be able to supply the maximum quantity of water required for the lands under it, and the level of the sill should be such as to admit of irrigation being carried on, until the crops are fully matured. Every sluice should be regulated from a platform at or above F. T. L.

2. Selection of Site :—(1) A site, which will prove as the best for the safety of the dam itself, will be at the centre of a saddle or depression in the natural ground, across the central line of the dam, as then the embankment will settle symmetrically on both ends. There will thus be no tendency for any crack or slip : and any leakage, which may occur, will naturally find its way at once, to the centre of the depression and will not lubricate the base of the dam beyond it.

(2) A site which will enable a masonry culvert to be entirely in cutting and to have hard foundations, so that the settlement of the dam will not affect the culvert, and enable the puddle trench line to be crossed safely. This is mostly for an earthen dam.

N. B.—A site which gives the most suitable alignment for the canal should be selected, but this is of minor consideration as the first section of the alignment is comparatively short in length.

(3) A site on the bank, opposite to that on which the waste weir is situated so that the floods from the waste weir will not have to cross the channel from the outlet.

The worst site is where the sluice is situated on the steep slope of a gorge. Here on the water-side, the earthwork tends to settle away from the outlet, and the subsoil percolation lubricates the base.

3 Number of Sluices: Outlets are a source of weakness in a dam, and their number should be restricted, as they form a discontinuity in the earthwork of the dam.

Where there are two channels on the same bank, one of them will be at a higher level and its construction will not be objectionable. Where the two channels are on opposite banks, it should be verified, if one outlet channel cannot be made to serve both by bifurcating the main channel from it at some distance from the

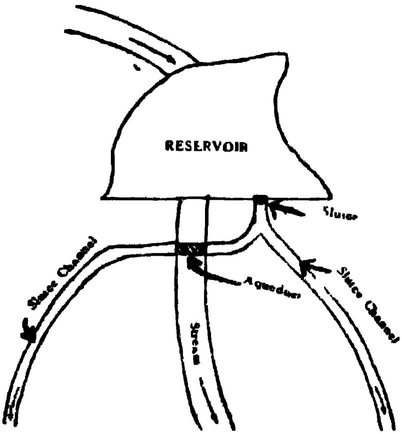


Fig. 151.

dam and carrying one of them by an aqueduct across the main stream. See Fig. 151.

The greater the number of sluices, the weaker is a dam; where land is to be irrigated on either side of a valley, it is better to have two sluices.

Design: The working of the sluice gate is influenced by

1. design of the sluice.
2. design of the gate.

Due to the location of the gate, an abrupt change in the velocity and the static head takes place. This causes disturbance in the sluice and often produces

1. negative pressures and
2. cavitation in the sluice, below the gate.

The entrance to the gate should be properly shaped, otherwise it will cause disturbance which persists through the sluice.

The surface of the sluice or the gate should be quite regular otherwise it produces harmful effects.

N. B.—The above defects are reduced by the proper design of the waterway.

The area of the sluice should be reduced gradually, between the entrance and the control or gate.

Note :—The base or bottom of the gate should be well protected with a lining of cast iron or steel.

4. Size of Sluice : This depends upon the area irrigated. Where the outlet consists of a culvert under the dam, it is calculated by the formula for a drowned orifice,

$$Q = \frac{2}{3} cl \sqrt{2g} \left(h_1^{3/2} - h_2^{3/2} \right)$$

where, h_1, h_2 are the depths to the lower and upper sills of the opening below the water surface. As is usual in the case of high reservoirs, with water standing near the F. S. L., the depth of water is very great, compared to the height of the opening. It will be sufficiently correct in practice, to use the simple formula for the discharge from a small orifice.

$$Q = CA \sqrt{2gh} \text{ and } A = l \times h$$

where, h is the head measured to the centre of orifice, A area of orifice, l = breadth, h = height.

(The results in the two cases differ only by 6%).

It must be remarked, however, that the size of the opening must be made large enough to give the (maximum) full supply

discharge of the canal or slightly lower, at the time when the head is a minimum, that is, when the water stands only 2 to 3 ft. above the outlet sill.

In special cases, very often, the outlet takes the shape of a pipe with or without a culvert.

N. B.—Sometimes there are 2 lines of pipes for each outlet, each controlled by a valve at the extremities. Two pipes are provided so that one may be in use, while the other is undergoing repairs, hence each pipe should be large enough to give the full required discharge. This is the case in water-supply projects.

5. Level of Sluices: This is fixed by taking into consideration the level of the land to be irrigated, but generally this need not be taken as essential, because the full extent required can be easily commanded by the extension of the sluice channel. The more important considerations are

(1) Space should be provided below the outlet sill level for the accumulation of silt—a rough rule for this purpose is, to allow 1/10 of the original storage below the sill level. (The capacity of a tank in the lower level is very small and as such it is not worthwhile to incur extra expenditure in tapping the lower part of the storage.)

(2) The lower the sill, the greater will be the insecurity of the outlet and greater the expense.

The higher the sill, the channel gains quicker command and shorter and cheaper will be its course.

6. Subsidiary uses of the Sluice:—Sluices are sometimes used for other purposes than their logical use of leading the water to the irrigated lands.

(1) To reduce silting up of the reservoir bed, sluices with a number of outlets are sometimes provided. These are opened during the early monsoon which brings the most silt, and thus accumulation of silt is prevented.

(2) During the construction of a new tank, for the final closure in the river portion a flood gap is necessary. The sluice is taken advantage of, to allow a portion of it (the flood),

(3) During floods, the sluices are open, and thereby the rise of the water level is made gradual and not sudden.

(4) During heavy floods, to lower the water level in the reservoir, a number of sluices are provided in some dams to discharge the flood waters.

This is a dangerous risk, unless the establishment takes immediate action at the time of floods. It is better to depend upon the weirs only than on controlling the sluices.

7. Types of Outlets or Sluices :—There are different types of outlets in use in the country. The types used depend upon the locality, that is, the nature of the materials and labour available.

(1) The oldest type of outlet consists of a barrel or culvert, to lead the drain from the reservoir to the lands, with arrangements for controlling it at the front or water side.

The type is developed mostly in the controlling arrangement.

The old method of controlling was by the construction of an inner cistern in the bed of the tank, a little distance from the front toe and by providing a plug with a rod to control entry of water into the barrel. (See Fig. 152).

OLD TYPE SLUICE

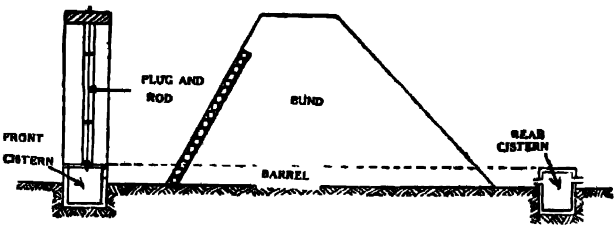


Fig. 152.

This type necessitated a long bridge from the top of the bund to the site of the controlling arrangement and the maximum length of the barrel or tunnel extending from front to the rear toe.

The tunnel or the drain was built of stone slabs, with a flooring or covering of slabs only. Also, the dimension of the vent was generally small being $1\frac{1}{2}$ ft. × 1 ft. or even 1 ft. × 1 ft.

The *disadvantage* in the method was, either the choking up of the slab drain due to the insufficient fall, or leakage in the sides owing to improper construction. The work could not also be easily inspected.

(2) An improvement over this type is the "*Belgaum type of outlet.*" Here a head wall with controlling arrangements is constructed just at the front toe itself. Even here, a foot bridge is necessary to control the plugs. The slab drain or the tunnel is also small, being 1½ ft. × 1 ft. or 1 ft. × 1 ft. The water from the reservoir enters directly into the barrel, and at this junction a cast iron T pipe is fixed for the plug to operate. See Fig. 153.

TANK SLUICE—BELGAUM TYPE

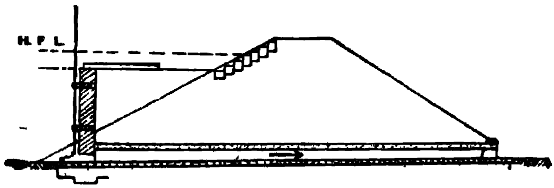


Fig. 153.

(3) A further improvement on the Belgaum type is the *Dharwar type outlet.* Here the head wall is constructed practically near the top of the bund (at the junction of the front slope and the F. T. L.) There is also a curtain wall provided in the front with wing walls for the head wall which thus forms an outlet chamber. There is also a rear head wall provided, a little distance inside from the rear toe. The dimension of the slab drain is 1½ ft. × 2 ft. and the head walls being near to each other. (See Fig. 154), the length of the barrel is much reduced. This

TANK SLUICE–DHARWAR TYPE

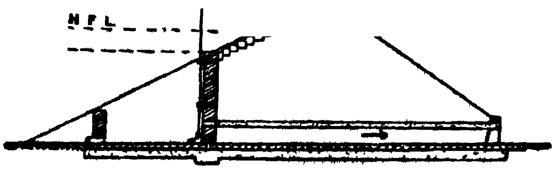


Fig. 154.

facilitates easy inspection and repair. Also, the inlet chamber prevents silt entering in. As in the Belgaum type, a plug and rod arrangement is also made at the head.

(4) The sluices constructed now-a-days are of the type known as '*culvert under the dam*' or bund.

This is the cheapest form and adopted generally in all places, The culvert consists of either a masonry slab drain or a concrete circular tunnel or a cast-iron or hume pipe, on the water side. The main head-wall is constructed with its top generally 1½ ft. to 2 ft. above H. F. L. On the rear also, a head wall with wings is constructed. (See Fig. 155.)

OUTLET CULVERT UNDER THE DAM

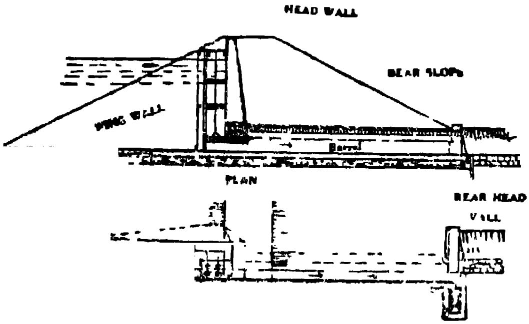


Fig. 155.

There is no *cistern* at the front. Only two wing walls are constructed butting the main head wall and to conform with the front slope of the earthen dam.

To lead the water from the reservoir into the sluice, an approach channel is excavated from the sluice to the bed of the reservoir, the bottom of the channel being the sill of the sluice. The size of the culvert is made neither too small (which will be difficult for inspection or repair) nor too big (in which case it may give way due to the weight of the dam.) The size is generally made for a man to go easily into the tunnel for inspection, a minimum size of 2 × 3 ft. is required, A fall of 1 in 100 for the bed of the drain is allowed.

Defects in Type No. 4 and Remedies Suggested to Overcome the Defects

Defect	Remedy
(1) Due to the settlement of the dam, the tunnel may be disturbed.	This is prevented by careful execution of the work. The foundation for the culvert should be taken below the ground surface to hard soil. The front and the rear toes of the embankment should be well supported and prevented from sliding, by proper design of head walls.
(2) There is a likelihood of settlement and the formation of crack at the junction of the culvert with head wall.	The culvert should be over lapped by concentric arching to carry the weight of the masonry of the head wall.
(3) There is a liklihood of a fracture at the junction of the puddle trench and the tunnel.	The foundation of the culvert should be carried below the bed of the puddle trench, or in the alternative, a concrete trench may be constructed so that the culvert and the concrete trench will bond together well.
(4) Leaks generally occur at the junction of the outer surface of culvert and the earthwork.	This is prevented by the provision of a thick coat of clay over the masonry or the culvert ring and further strengthened by the construction of stancheon rings round the culvert in three or four places along the line of the tunnel.

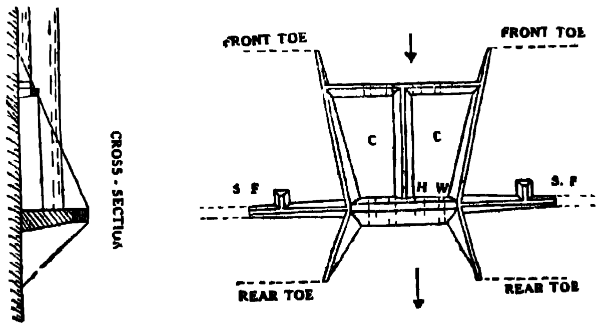
8. Head Wall in the Centre Line of the Dam:—In this type, the main masonry wall is built, as said above, in the centre line of the dam. It is provided with openings for the outlets which are controlled by valves worked by lifting rods and capstans.

Precautions should be taken to prevent the creep of water along the upstream wings by the construction of stancheon walls

or forks, long enough to prevent the creep, and also with cross walls battered on all sides for the same purpose. The head wall generally acts as an ordinary masonry dam, and accordingly (should be designed as such, and it should be) founded on solid rock. The wing wall also should be similarly founded on solid rock. If the wing walls are close together, the section of the main head wall may be reduced since they act as abutments.

The cross wall is constructed in the centre to form an inspection chamber. See Fig. 156.

SLUICE FOR RESERVOIR—HEAD WALL IN THE CENTRE LINE OF THE DAM.



S. F. Staunching Fork. C. Inspection Chamber

Fig. 156.

Sometimes a partition wall is constructed in the chamber, so that half of it may be used at a time.

Advantages :—The advantages of this typs are

(1) The work can be easily inspected and any necessary repairs done easily.

(2) As the outlets can be large in number, the surface of the reservoir can be kept low in the case of a new earthen dam so that time may be allowed for the consolidation of the dam under the minimum amount of infiltration and this allowed for some years until the dam settles and the irrigation develops.

(3) During the early parts of the monsoon, when the floods have most silt, the outlets can be easily opened and the deposit of silt prevented.

(4) In the case of an accident, the level of the reservoir can be lowered rapidly and thus the discharge through the waste weir reduced.

Disadvantages :—The disadvantages of this type are

(1) It admits the water to the heart of the dam.

(2) It is very costly.

9. Outlet Tower :—When supply to the lands is to be regulated properly, the outlets for this regulation have to be situated sometimes at different levels. For this purpose, the best arrangement is the construction of an outlet tower.

The structure is generally made circular throughout, except at the downstream interior, where it is made rectangular to facilitate the easy working of the gearing rods for controlling the water supply. The circular form is the strongest and the cheapest. To make the structure watertight, it is generally built of cement. For inspection purposes, either a ladder or permanent vertical steps are provided.

10. The Dam slope outlet:—This type is suited only for small reservoirs, where the expense of a masonry head wall is to be avoided. Low masonry chambers are constructed along the inner slope of the dam, in which one or more openings with pipes are made in the sides, the mouth of the pipes being controlled by means of valves worked by handles laid along the slope of the dam. The ends of the outlet pipes are turned through a quadrant (so that the

DAM SLOPE OUTLET

SECTION

PLAN

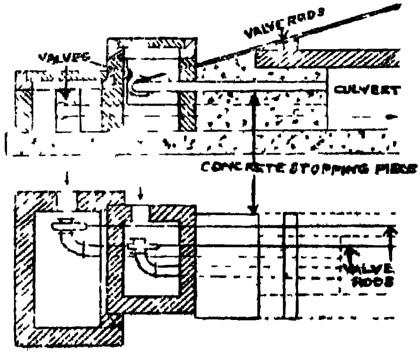


Fig. 157.

valves may be circular and not elliptical, as would be the case if the mouth of the pipe were straight). The lifting rods of the valves are made to pass through guides fixed to stones, supported on the slope of the dam, and are worked by gearing rods placed at top. (See Fig. 157).

11. Tunnel beyond the dam proper (in ridge cutting) :— This type of sluice outlet is very safe for the dam, because it is independent of it and cannot affect it in anyway.

As seen from Fig. 158, a tunnel is made in the hillock or ridge, and is taken from the reservoir towards the rear. A leading or approach channel is necessary for the water to enter into the tunnel. The alignment for the approach channel is made at right angles to the dam for some distance and is then taken into the bed of the reservoir. At the rear also, it is carried on at right angles for some distance and then taken in contour.

The *disadvantages* of this type are

(1) If the tunnel is long, the cost is prohibitive.

(2) If the excavation is in fissured rock, there is a likelihood of leaks occurring.

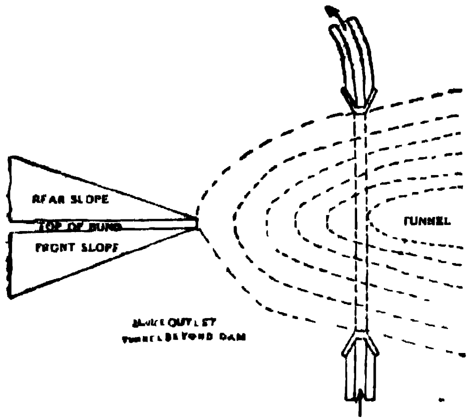


Fig 158.

(3) The supervision during construction and afterwards is not easy; hence the headwall (or the *portal* wall, as it may be called) should be built solidly with good connection into the sides of the rock.

12. Open cut:—Another type of sluice outlet s also outside the portion of the dam proper, but here, instead of the tunnel, an open cut is made. To regulate the supply, a head wall is necessary even here, also the necessary approach and lead-off channel (See Fig. 159).

OPEN CUT SLUICE BEYOND DAM

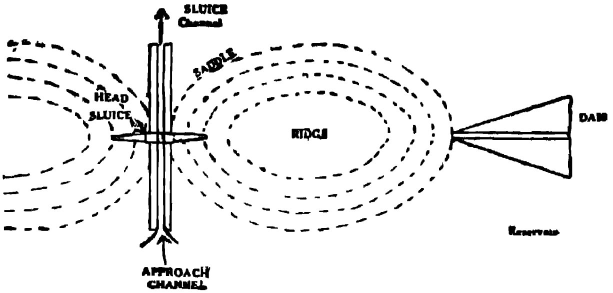


Fig. 159.

The *advantage* of this type is, that the whole work being in the open, it is easy for inspection and repair. The site selected may with advantage be taken in a natural dip as shown in Fig. 124.

The rock penetrated through should be sound, otherwise there may be percolation or leakage, in which case, a lining with cement concrete may be required.

This type can be used where the depth of the cutting is not much, and where the full supply depth is also small.

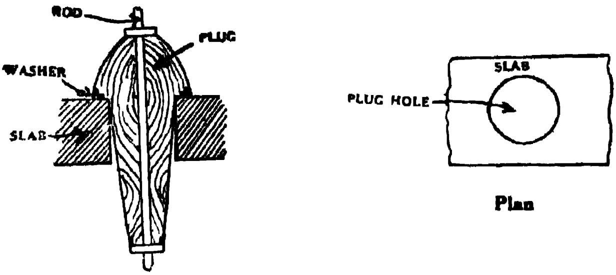
13. Controlling Arrangements:—*Plug and Rod.* The earliest method of the controlling arrangement was the pole and the plug, the pole being either of Bamboo or sometimes of wood, and the plugs mostly of wood, (Babul or Teak). Recently, the pole is of iron with screwgearing arrangements, and the plug is still of wood as it is more watertight. Sometimes cast iron plugs are also used for some tanks. Figs. 160 and 161 show the details of the wooden and the iron plugs.

The merits and demerits of a wooden plug and rod are noted below:—

Advantages :—(1) It is simple.

(2) It can be easily worked.

WOODEN PLUG



SECTION

Fig 160

(3) It can be easily inspected.

(4) It enables the contents at the bottom of the reservoir to be drawn off with the minimum head.

TANK SLUICE

IRON PLUG

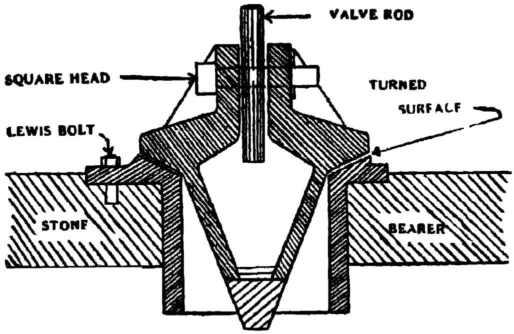


Fig. 161.

Disadvantages :—Under a great head of water, on account of its form, it would be difficult to work it.

(2) It is subjected to much vibration.

14. Stoney's Sluice Gate. This is a gate which bears at its ends against a train of rollers, and it is usually hung (on both sides) by chains, which pass over pulleys to a counter-weight and this helps to manage the gate easily. The roller train or cradle is also hung in the groove by a cable and it is counterweighted.

As the side of the gate bears against the rollers when the gate is moved, the rollers revolve and the whole train of rollers rises and falls with the gate.

The gate is operated by gearing on the supporting sheaves by hand cranes or travelling electric motors. Vide Fig. 162.

The friction induced is pure roller friction, which between smooth surfaces is very small. The lifting power is the weight of the gate plus friction of the lifting apparatus.

STONEY'S GATE

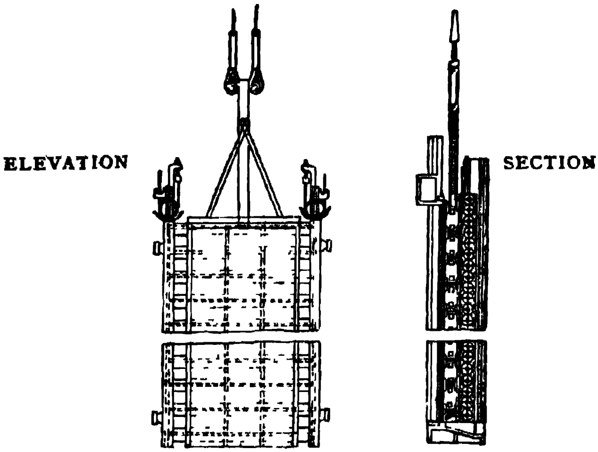


Fig. 162.

Taken from Krishnaraja Sagara Hydraulic Research Station Publication 2.

Stoney's gates are used where

(1) the breadth of the span of the shutter is large; that is, where the breadth is about 15 to 40 ft. and the height is from 6 to 8 ft.

(2) the span or the breadth is small, say 6 to 7 ft., and where the head is great, so that the required quantity can be discharged easily, and the height of the vent adjusted (as the head is great to suit the discharge).

(If the depth of water over the sill of the sluice is more than 80 ft., it is not advisable to use the shutter safely).

Types: The gates are of two types (1) with fixed rollers and (2) with free rollers.

Fixed Rollers :—In this case, it is difficult to keep the rollers in good running order, especially when they are immersed in water containing a good amount of silt. Again, with the fixed rollers, axle friction is developed very rapidly and the roller may even fail to rotate after some time. The fixed roller type is not as efficient as the free roller type.

Free Rollers :—This type is not affected by muddy water, and it does not become rusted; also it is not subject to axle friction. The efficiency of this type is more than that of the fixed type.

Stoney's Gate :—The permissible loading for the rollers is given by the formula.

$$P = C \times L D$$

where P is load in pounds

C is a constant

L is the length of contact of roller in inches

D is the diameter of the roller in inches.

Defects of the Stoney Gate and Remedies suggested*

(1) The roller frame shears off between $3\frac{1}{2}$ to 5 ft. height from the bottom; as such, the operation of the gate becomes difficult.

The roller forms a wedge and gets stuck up, due to the bent roller frame.

The rollers being exposed to water action when the gate is opened partly, they get bent very soon.

Remedies :—(a) Provide a shield plate to the roller frame with a gap for the side girder to move up and down.

(b) Increase the thickness of the side plate of the roller frame from $\frac{1}{2}$ to $\frac{7}{8}$ in. thickness.

(c) Increase the width between the side plates.

(d) Provide the lower-most 6 rollers with springs to prevent them from turning easily by water action.

(2) Leakage when the gate is closed. It is seen that at full reservoir level in K. R. S., the leakage through the gates was 7·5 cusecs.

* Taken from the Journal of Hydraulic Research, Mysore.

Remedy:—Introduce a circular iron or brass at the sides of the gate working loosely at the sides.

(3) Corrosion of the gate and wire ropes. Though painted with anti-corrosive bituminous paint, the skin plate becomes pitted. A number of pits ½ in. deep developed on the skin plate. This was more seen at the top and the bottom 2 ft. of the height of the gate.

The wire rope passing over the pulley and fixed near the lintel, is subject to the action of water splashing over it. Hence it is also subjected to corrosion. This is a common defect found everywhere. The girder at the rear throws scales year by year.

The Remedy is to raise high the point of suspension of the wire rope.

(4) Bumping of the gate: This is due to the formation of large cavitation areas which set up vibrations.

Remedies :—(a) To give a correct bell-mouth opening to the gate, and

(b) To provide steel damper plates.

Stoney's sluices are placed 6·4 ft. inside from the face of the dam. The bell-mouth entry is given at the sides and top, the angle of divergence being 10°, and the bottom sill is made flat.

The coefficient is affected by the position of the gate, with reference to the front face of the dam, in high head sluices.

15. The following **conclusions** are based on the results of the sluice coefficient observations of Krishnarajasagara Dam.

"(1) That for a given vent opening of a sluice, the coefficient increases with the increase in head under which the discharge takes place, and tends to become constant at high heads.

(2) That the effect of opening the adjacent vents is felt only at low heads (for higher openings and not at high heads) whatever the size of the opening may be.

(3) That the coefficient for a given reservoir level decreases with the increase in the height of the opening, and again increases as the height of the opening is increased, the minimum coefficient occurring when the vent is opened nearly to half the height.

(4) That a bell-mouth entrance increases the discharge through the sluice.

Also (1) For a given opening, the coefficient increases as the head increases, until in the neighbourhood of the 10 meters head, it becomes constant.

(2) For small openings (1·5 and 2·0 metres), there is not much difference between the coefficients for the same head.

(3) For small heads, there is a progressive decrease of coefficient, as the size of the opening increases. For heads greater than 3 metres, this effect is reversed and the coefficient increases with the increase of the sluice opening.

The coefficient differs with the head and the area of the opening. For river sluices, it varies between 0·7 to 0·9."

WEIRS

1. **Surplus Works**:—As said already in a previous chapter the constituents of a storage reservoir are (a) the dam proper, (b) outlets or sluices and (c) surplus works or weirs. Surplus works are structures built in a reservoir to store the water to a certain level (F. R. L.) and when in floods, the structure should safely pass the surplus water over it.

2. **Location of Surplus Works**:—The surplus works in any reservoir may be provided,

(a) at the flanks,
(b) in the body of the dam,
(c) over the dam, either in part length, or in full length.

If the surplus work is provided either at one flank or both the flanks, it is called by the common name '*weir*' or '*waste weir*'. It is in the shape of a trough or a chute. Some call it *trough spillway* or *chute spillway*. This type is mostly used for earthen dams and also, for masonry dams; but big masonry dams may contain in addition to the above, either the one or both types of the other surplus works also, (i. e. *b* or *c* or *b* and *c*).

3. **Site for the Weir type of Surplus Work**:—The site for a waste-weir of an earthen dam must be selected very carefully, as on it depends the safety of the dam. The best site is in a separate saddle, outside the length of the main dam, so that the surplus discharge will pass safely away from the dam, in an independent natural stream. The worst site is an artificial cut at

the flank with embankments on both sides to protect the flood discharge from endangering the rear of the bund.

4. Types of Weirs: There are various types of weirs for reservoirs, big and small, depending upon the locality, the nature of the ground, and the depth of discharge over the crest of the weir

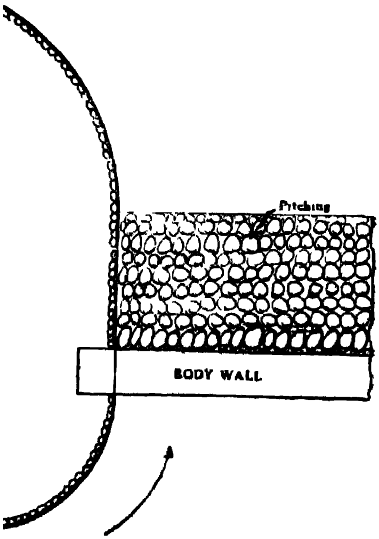
The types in general use are:

1. Flush escape
2. Body wall with sloping or horizontal apron
3. Weir with curved sloping apron, clear overfall weir
4. Weir with water cushion
5. Weir with vertical drop and horizontal apron

(1) *Flush Escapes*:—When the ground level on either side of the weir and the F. R. L. coincide or differ slightly, and when the soil is hard and also the depth of discharge is small, flush escapes are adopted.

Their design is very simple; only the ground in front and rear should be levelled and left as it is. If stones are available near by

FLUSH ESCAPE



Plan

Fig. 163.

(in plenty) pitching for about 6 ft. width should be done to the rear; and if stones are not available, good turfing should be done for a width of about twice the length of the weir. Fig. 163.

(2) *Body wall with sloping or horizontal apron* :—When

BODY WALL WITH SLOPING APRON

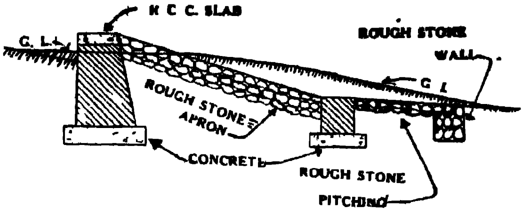


Fig. 164.

the fall from the weir body wall is small, the design adopted is a body wall with a sloping apron or a body wall with a horizontal apron at a lower level. (See Figs. 164 & 165).

WEIR WITH HORIZONTAL APRON (DROP 3 FEET)

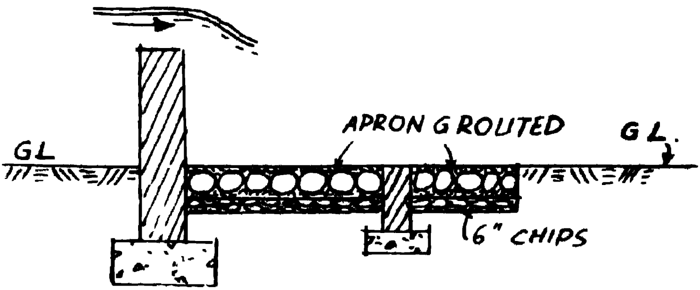


Fig. 165.

Steeped Apron :—This is suitable where the foundation is soft and the drop about 4'–6' (Total). The thickness of the apron is made $1\frac{1}{2}$ ft. and it is laid on a bedding of small chips 6 in. in thickness. The top surface of the apron is grouted.

(3) When the fall is great, the designs adopted are (a) weir with curved sloping apron, and (b) clear overfall weir.

(a) *Weir with Curved Sloping Apron*:—The discharge at the foot of the fall is in a horizontal direction, and the velocity here closely approximates to that of water falling vertically from an equal height. There is no impact here. The curved apron has to be efficiently constructed with concrete, See Fig. 166. The curve should be struck from a centre in a line in the upper or inner face of the retaining wall.

WEIR WITH CURVED SLOPING APRON

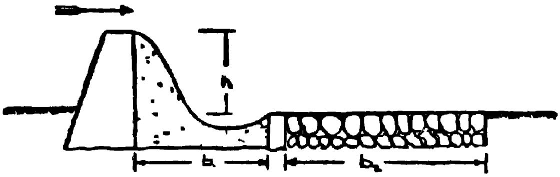


Fig. 166.

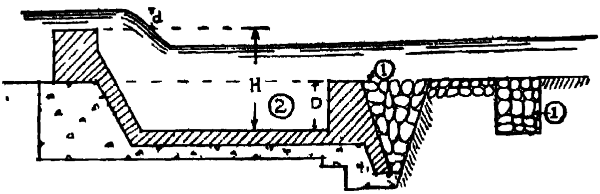
Width of apron $b_1 = 2\frac{1}{2}$ times the drop $= 2\frac{1}{2}$ h.

Rough stone apron beyond (width or breadth) = 3 times the drop. i. e. $b_2 = 3$ h. Vide Fig. 166 above.

(b) *Clear Overfall Weir*:—This is one, which has its crest above the surface level of the tail channel discharge, when the reservoir is at H. F. L.

Foundation:—The action of the overfall weir is very severe on the foundation. The weir wall must be founded quite securely. The best foundation is one of solid unfissured, compact rock, on which the weir will be founded and deepened, if necessary, to at least 3 to 5 ft. depth, according to the nature of the rock. In the

WEIR WITH WATER CUSHION



(1) Boulders Grouted with concrete (2) Water Cushion.
Fig. 167.

case of an ordinary soil, the foundation must be protected by a water-cushion. This lessens the horizontal velocity of water.

(4) ***Weir with Water Cushion*** :—The design of this type is shown in Fig. 167. Here the depth of water-cushion is given by

$$D = c\sqrt{H^3 d}$$

where, D is the ***depth of the cushion*** below the top of the retaining wall;

c is dependent upon the material used for the floor of the cushion, it is 0·75 for compact stone, 1·25 for moderately hard brick;

H is the height of the drop;

d is the maximum depth of water over the crest.

The *width* of the floor of cushion depends on the section of the floor. It is between $6\sqrt{d}$ and $8\sqrt{d}$.

Another formula usually adopted for water cushion is

$$X = H + \sqrt[3]{H}\sqrt{d}$$

$$Y = \tfrac{4}{3}\sqrt{z} \times H,$$

where, X = depth of water cushion

H = height of water over crest of weir.

Z = height of weir wall.

d = difference of water level.

and Y = width of apron. See Fig. 168.

WEIR WITH WATER CUSHION

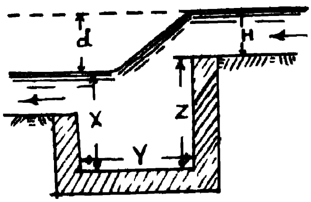


Fig. 168.

(5) ***Weir with Vertical Drop and Horizontal Apron*** :—In this design, shown in Fig. 169, the masonry apron is formed of fairly regular blocks of stone or cement concrete.

The width of apron x varies from $6\sqrt{D}$ to $8\sqrt{D}$, and the thickness from 1/5 (H + D) to 1/4 (H + D),

WEIR WITH VERTICAL FALL AND HORIZONTAL SOLID APRON

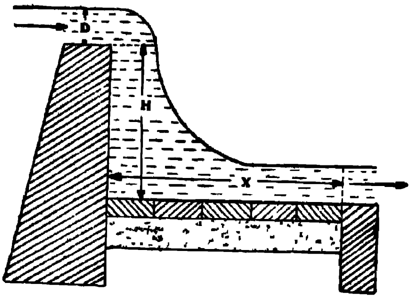


Fig. 169.

Section of Upstream Edge of the Weir Wall.

The stone work for the apron should be bedded in concrete and should not be less than 1 ft. The depth of the foundation of the retaining wall and of the returns of the rear wings should be considerable. A rough stone apron of good width and thickness should be provided to protect the foundations.

N. B : The apron type is not much recommended due to scour at the bottom. Also there is extra cost for a flat apron. This type is only used where the bed is hard and the stones are cheap.

5. Design of a Weir : Regarding the design of a weir two things are important. They are

(i) The length of the weir,

(ii) The section of the weir.

To deal with the first item i. e., the length of the weir, what is required is the maximum discharge over the weir.

This is generally found by using either Ryve's formula or Dicken's formula or by actual gauging.

$$Q = CM^{3/4} \text{ Dicken's; or } Q = CM^{2/3} \text{ (Ryve's.)}$$

Where the reservoirs are constructed in series, i. e., one below the other, a good quantity of the discharge from the catchment is stored up, in the above tanks. Hence where a tank catchment consists of both *independent* and *combined catchment*, the above formula is modified as

$$Q = CM^{2/3} - cm^{2/3}.$$

where c is generally taken as 1/5 of C,

and m is the catchment area of the intercepted catchment.

Combined catchment is the total area of the whole catchment above the tank. (Independent + Intercepted).

PLAN SHOWING CATCHMENT AREA

FOR CALCULATION OF TANKS IN SERIES.

TOTAL DRAINAGE AREA = INDEPENDENT + INTERCEPTED

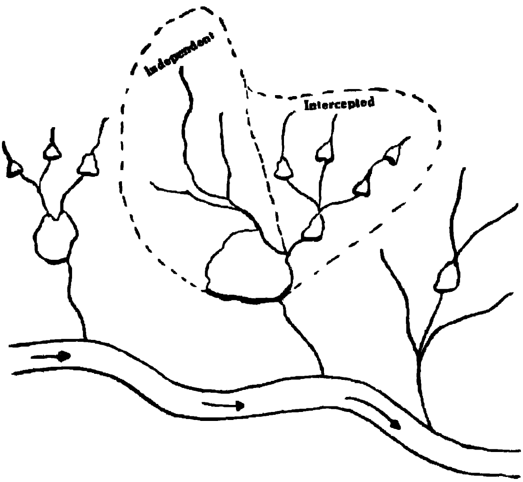


Fig. 170.

Independent catchment is the area which drains only into the tank under consideration.

Intercepted catchment is the difference between the combined catchment or the total catchment and the independent or the free catchment area. (See Fig. 170).

6. Tank Weir: The formula for discharge in a rectangular notch is

$$Q = \frac{2}{3} c\, l\, h \sqrt{2gh}$$

The weir of a tank is a rectangular notch, hence the above formula applies to it. Here c, the constant, is taken as 0·577 or $\frac{1}{\sqrt{3}}$ as per Professor Unwin. This gives,

$$Q = \frac{2}{3} \times \frac{1}{\sqrt{3}} \times l\,h\,8\sqrt{h}$$

$$= \left(\frac{2}{3} \times \frac{1}{\sqrt{3}} \times 8\right) l\,h^{3/2}$$

$$= 3{\cdot}1\,l\,h^{3/2}.$$ This is the formula generally used.

7. Reducing run-off from square miles per hour, to discharge in cusecs: By this method, a similar and more direct formula applicable to all cases is obtained.

$$Q = Md \times (\text{a constant} = 645)$$

M in square miles = M × 640 × 4840 × 9 sq. ft.

d in inches = $d/12$ ft. per hour

but Quantity per hour = Quantity 1/60 × 1/60 per second

$\therefore\ Q = Md \times 640 \times 4840 \times 9 \times 1/12 \times 1/60 \times 1/60$

$\therefore\ Q = Md \times 645$

where M is catchment area in square miles

d is the run-off in inches per hour

Q is the discharge in c. ft. per second.

8. Broad-crested Weir ($c = 0{\cdot}577$) :—This is not correct for all widths and all depths of discharge. c decreases with increase in width, and increases with increase in depth. If the head extends 1½ to 2 times the crest width, the jet may jump clear of the crest (sharp-crested weir).

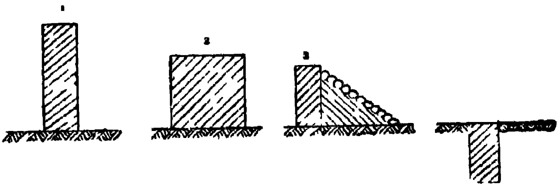
The formula, for broad-crested weir is

$$Q = CL\sqrt{h^3}$$

The value C for different types (See Fig. 171) of an ordinary weir, is noted below.

Top widths of wall	Less than 3ft.	Greater than 3 ft.	Sloping	Flat
C taken	0·625	0·562	0·5	0·437

DIFFERENT TYPES OF WEIRS FOR TANKS



(1) Weirs with crests up to 3 ft. wide
(2) Weirs with crests over 3 ft. wide or with dam stones
(3) Rough stone sloping escapes
(4) Flush escapes. (flat)

Fig. 171.

In curved weirs, due to the rise in head, more water is forced towards the head regulator of the reservoir. A curved weir for a tank with sheer over-fall is therefore preferred. (Fig. 172).

9. Waste Weir Discharge:—Calculations usually applied to ordinary tank weirs are given below. The discharge, when there is no appreciable velocity of approach, is,

$$Q = \tfrac{2}{3}\, c\, l\, h \sqrt{2gh}$$
$$= \tfrac{2}{3}\, c\, l \times 8{\cdot}02 \sqrt{h^3}$$

WEIR (CURVED IN PLAN)

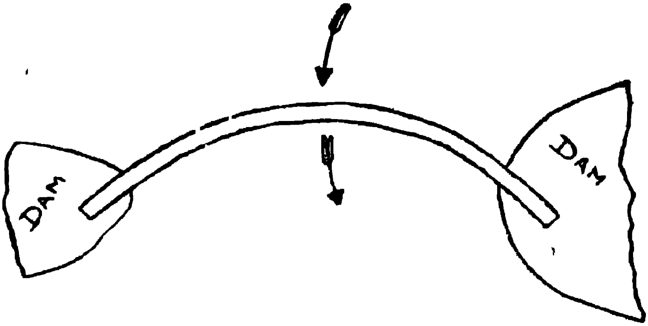


Fig. 172.

where c is a coefficient, the value of which varies with the form of the weir but seldom exceeds 0·65;

h is the difference between M. W. L. (or H. F. L.) and F. T. L. & l is the length of the weir.

If $c = 0{\cdot}6075$, discharge $= Q = 3{\cdot}25\, l \sqrt{h^3}$,

10. Over-Flow Dam :— (*Spillway Dam*) :— In addition to the usual design of the dam proper, provision should be made, in the design of this type, for the thrust and dischage of the over-topping water. Rounded crest is the best form.

The water falling creates erosion on the rear toe; hence this should be specially seen to. To absorb the shock, the toe is paved with concrete or a water cushion is created.

Ogee Weir : A form for the overflow dam is an *Ogee Curve.* Here the downstream face of the weir is shaped to a parabola, to coincide with the flow of the falling water, and a reverse curve is given at the toe, to guide the water from a vertical to a horizontal course when leaving the toe. (Vide Fig. 173).

WEIR WITH OGEE CURVE

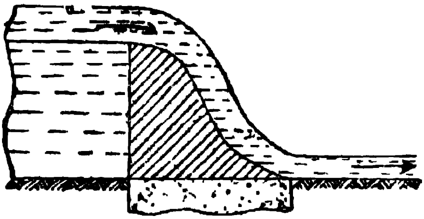


Fig. 173.

(This type is much in use in America for weirs on rock foundations).

The *advantages* of this type are :—

(1) Its construction and design are simple.

(2) The design is good, and there is no damage due to sudden floods, as it has a large discharging capacity. (When compared with the broad crested weir, it has 50% more of discharging capacity).

(3) Damage due to floating trees and branches is avoided.

(4) This is cheaper than a weir with automatic flood gates.

The *disadvantages* of this type are

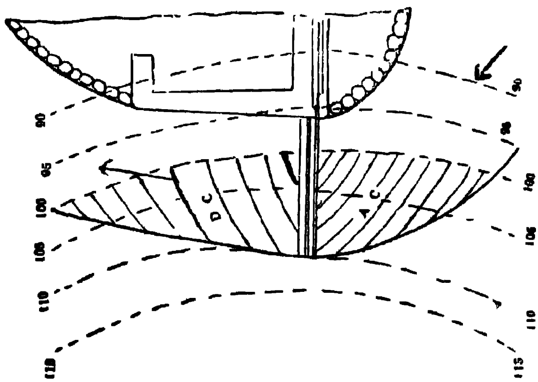
(1) There is a considerable waste of storage between the full supply level and the high flood level, when compared with the spillway with automatic flood gates, and as such, the dam should be built high, which means extra cost.

(2) Since the flood discharge can only be passed between the full supply level and the high flood level, the length required is large when compared with a syphon spillway.

(3) For low weirs, this induces scours in the margins and in the bed of the run-off channel, due to the very high velocity with which the water is projected horizontally from the foot of the fall and as such, this type is not much in use now-a-days. To remedy this, the velocity of the water moving downstream from the toe of the work should be reduced, which is done by providing a water ushion or an apron for the overfall.

12. Approach Channel :—The approach channel is usually excavated in a level with the crest of the weir, but where it is long and the water, in passing over it, will lose the head by friction of the bed, it is preferable to excavate it from 6 in. to 12 in. lower than the crest level.

The approach channel should have a clear and unobstructed



A C = Approach cutting

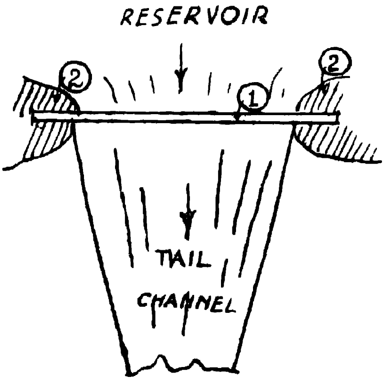
D C = Draft cutting

Fig. 174.

course of at least the full width of the waste weir; if not, water will head up and reduce the discharging capacity of the waste weir.

The amount of excavation for the approach channel may be considerably decreased, by curving it gently towards the tank side, as this will save the removal of the soil (in the portion hatched). It is usual to have the approach channel cleared as shown in Fig. 174 taking the centre of the circle as the beginning of the weir body wall and describing a circle with radius equal to the length of the weir wall.

13. Tail Channel or Draft Channel:—The tail channel must have a clear and unobstructed waterway, as otherwise water will head up. The channel might be gradually contracted at a certain distance towards downstream side as shown in the fig. 175a to diminish the quantity of cutting. This will also give a more defined course to the flood water.



1. Body Wall or Weir. 2. Dam
Fig. 175a.

The channel should start from the waste weir, whith the calculated bed fall. If this involves a large amount of excavation, the bed fall may be reduced, as soon as such reduction can be effected, without causing any heading up of the waste weir crest.

The tail channel should be led, as soon as possible, into the natural drainage line, so as to diminish the amount of excavation.

The tail water should be confined to the tail channel by means of

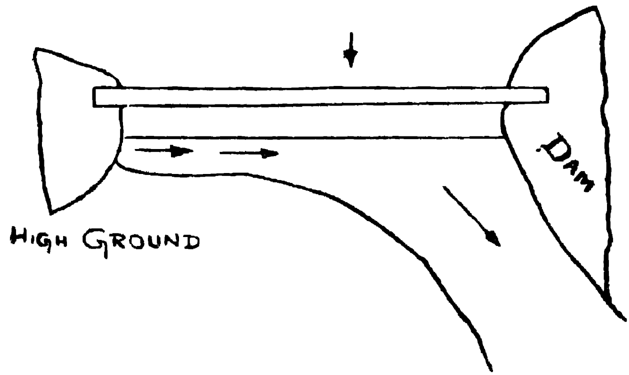
(i) Lining walls,
(ii) Protected slopes, and
(iii) Flood embankments,

until it can no longer cause danger to the reservoir works or to cultivated lands, buildings, etc.

In the case of a clear overfall weir, little or no excavation of tail channel is necessary, as the flood water flows down into the natural depression, which generally exists below the site of the weir.

Tail Channel Parallel to the Waste Weir: When the waste weir accommodation is naturally restricted in extent by high ground, a tail channel parallel to the weir at the start, is sometimes recommended. See Fig. 175b.

TAIL WATER



Running at the Start parallel to Body Wall

Fig. 175b.

Slope of Tail Channel: When the channel is in rock cutting, a slope of 1/100 can be given as the maximum. In fairly hard ground, the slope should not exceed 1 in 500; whereas, in ordinary soil, it should not be steeper than 1 in 1000.

When the natural fall of the country exceeds that of the given channel, it may be given a steeper fall, provided there is no retrogression of levels, and the waste weir itself is not affected by it.

14. Retrogression of Levels: The effect of high velocities is to deepen the channel in successive stages, by cutting back into the bed, where the soil is soft. This causes retrogression in levels.

Prevention: This could be stopped by the construction of a series of low curtain walls across the tail channel, and if possible, by founding them securely on rock. See Fig. 176.

In the absence of hard material in the bed, the curtain walls

RETROGRESSION OF LEVELS

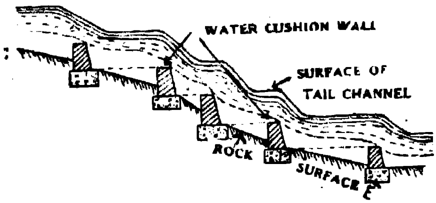


Fig. 176.

should be protected by aprons or stone rapids (See Fig. 177) or water cushions. These curtain walls hold up the water and form a water cushion.

RAPID FOR WEIR DISCHARGE

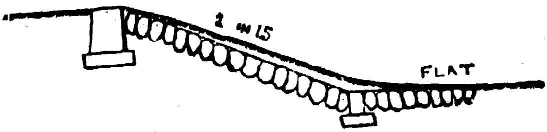
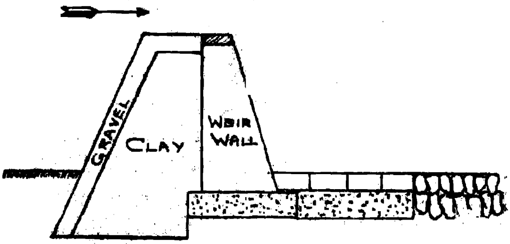


Fig. 177.

15. Special precautions to be taken in the construction of a weir:—

WEIR BODY WALL



Protection in front
Fig. 178.

(1) The body wall should not be raised by itself, because a drop will be formed, which may endanger the structure.

(2) To prevent any creep below the foundation, a layer of clay should be provided in front as shown in Fig. 178, which should be protected with a coating of gravel.

(3) The masonry returns at the sides of the weir should be built to a length of $2\frac{1}{2}$ times the height. AB, CD should each be equal to $2\frac{1}{2}$ height of body wall. See Fig. 179.

WEIR WITH RETURNS OF PROPER LENGTH

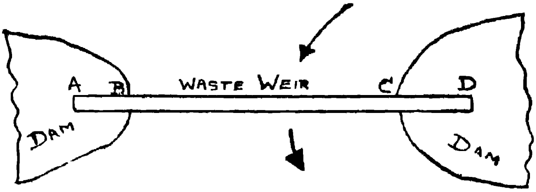


Fig. 179.

(4) The revetment at the sides and the apron should be grouted with mortar or concrete, if the depth of discharge over the weir is more than 3 ft.

(5) The apron should not be built as shown in sketch plan, Fig. 180, because a cross flow will be created instead of the usual flow of water at right angles to the weir line and this will endanger the structure.

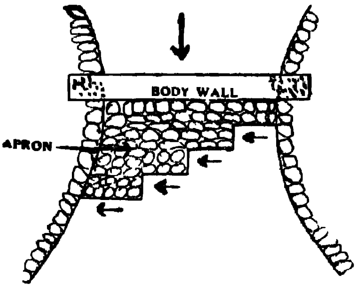


Fig. 180.

(6) The toe should be built flush or even below the ground level at the place

Questions.

1. Write a short note on Waste Weir.

(A. M. I. E., November 1952)

2. Explain what is meant by Retrogression of levels.

(Bom. Univ. B. E. Oct. 49)
(Mys. Univ. Final B. E. 48)

3. Write a short note on Water Cushion.

(Mys. Univ. B. E. Final March 50 and 47)

4. What is the main principle involved in the design of water cushion? Calculate the length and depth of the cistern necessary to negotiate a drop of 10 ft. in a place where the soil consists of soft rock.

(Mysore Univ. Final B. E. 48)

5. What is the function of a waste weir?

Discuss the conditions under which the following types of weirs are adopted:

(*a*) Masonry body wall with sloping apron.

(*b*) Water cushion.

(*c*) Horseshoe shaped body wall with clear overleaf.

6. (*a*) Sketch in plan and section the design you would adopt for the plug sluice of a minor irrigation tank. How is the diameter of the orifice determined?

(*b*) Explain briefly how the sluice is operated (1) when the water level is much above the level of the plug orifice and (2) when the water level is just below the level of the plug orifice.

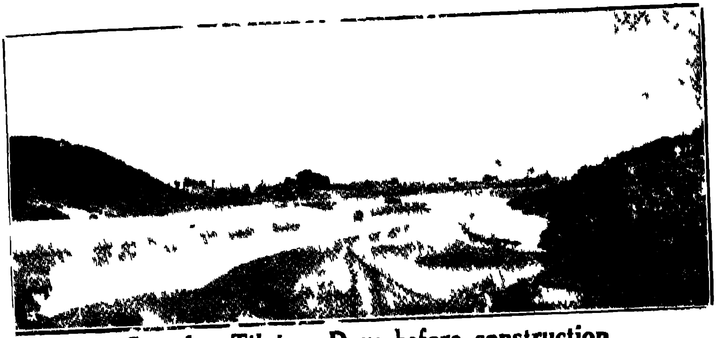
(Mysore Univ. B. E. Civil Sept. 1954)

7. Mention the different types of irrigation outlets that are commonly used in Mysore State.

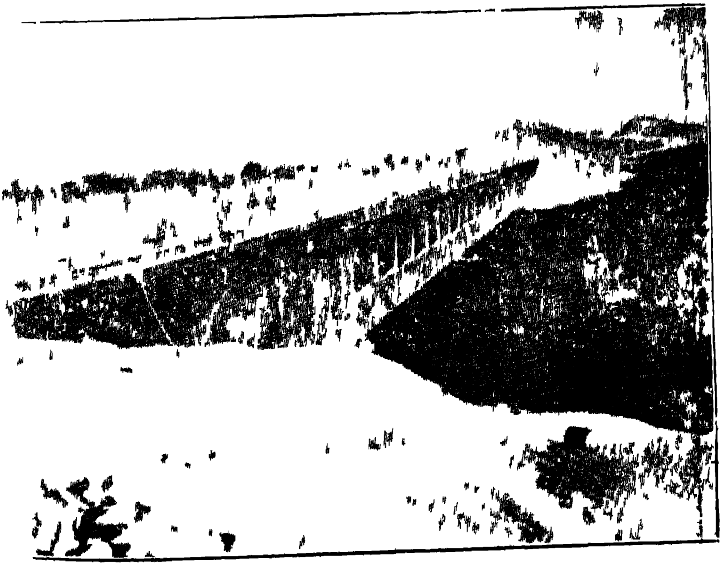
Design a sluice for a major tank to irrigate 200 acres of paddy cultivation with its sill 13 ft. below the F. T. L. and draw dimensioned sketches of such a tank sluice indicating the various parts thereon.

(Mysore Univ. B. E. Civil April 1955)

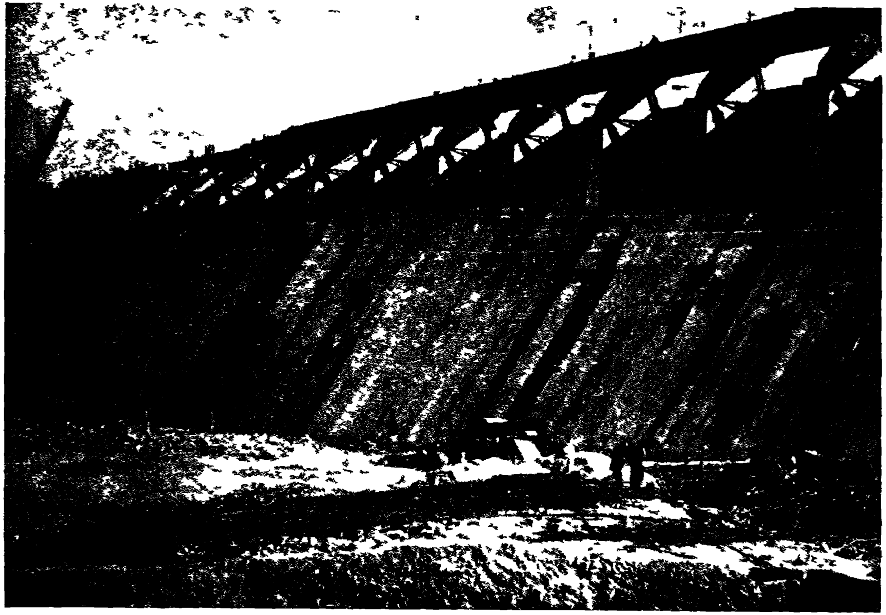
8. An earthen dam has a top width of 8 ft. at R. L. 47·00 and side slopes of $1\frac{1}{2}$ to 1 on the water side and of 2 to 1 on the



Site for Tilaiya Dam before construction



Tilaiya Dam



The Tilaiya Dam, Damodar Valley Scheme

rear. The M. W. L. of the reservoir is R. L. 44·00 and the F. R. L. is R. L. 42.00. The sill level of the sluice which is three feet below the ground level at that point is R. L. 10·00. Assuming that the diameter of the orifice of this plug sluice is 6 inches and all other details suitably, sketch (a) half plan at foundation and half plan at top, (b) longitudinal section through the tunnel of the sluice, and (c) cross-section of the tunnel. Dimension your sketches fully.

(Mysore Univ. B. E. Civil April 1957)

9. Write short notes on "Free Rollers"

(Gujarat Univ. April 1957)

CHAPTER XIII

RIVER HEADWORKS

Headworks:—Canals are provided with permanent works at the site of the weir, from where they take off. These works are known as *Headworks*.

SKETCH LAYOUT PLAN

SHOWING COMPONENT PARTS OF HEADWORKS

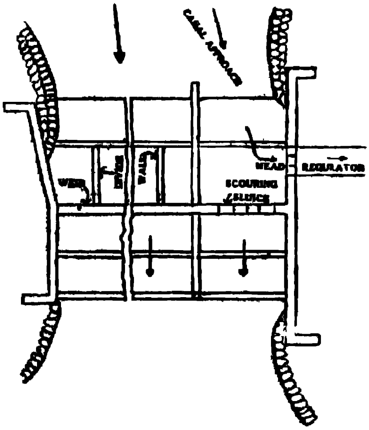


Fig. 181

1. Component Parts of Headworks:—(See Fig. 161) The headworks comprise,

(1) A weir, anicut, or dam across the river.

(2) A canal (with an approach channel).

(3) Head regulator of canal.

(4) Scouring or under sluices.

(5) Divide wall.

(6) River control works.

2. Selection of Site :—The following are the favourable conditions of site for headworks.

(1) The first requirement is *the permanence of the stream* channel, so that the stream will not leave the works after they are built (and not wash them out.) A straight length of the river, where the banks are well defined, and the velocity is fairly uniform and generally parallel to the axis of the bed, and where the cost of flood-banks and the training works is less, is recommended.

(2) *Good foundations*, especially rock or clay, should be met with. (If this is not practicable, a safe dam should be built with proper precautions.)

(3) *The approach* to the site of the headworks *should not be too wide*. This is to prevent the bed silt which would occur in a wide approach.

(4) The site must be able *to command the full area* proposed to be irrigated, with ample grade allowance for the canal.

(5) *The ground* on which the first few furlongs length of the channel is built, should be *of firm material*, and so high as not to be subject to the overflow of the stream, and the cost of the cutting of the canal, from the headworks to the point where it commences to irrigate, should be low.

(6) The location should permit of the *construction of regulating works parallel to the stream*, so that the current of the stream may have a clear sweep past the regulator and close to it, to facilitate sluicing away sand and silt that will threaten to pass into the canal.

(7) *Proximity to the materials of construction, and good communication with the sources of supply.*

3. Objects of Canal Headworks :—The objects of a canal headworks are

(1) To raise the water level in the river, so that it may command all the area to be irrigated.

(2) To control the entry of silt into the canal.

(3) To ensure the required supply into the canal.

4. Weirs. *Definition* :—A weir* or anicut is a rough stone or masonry wall, built across a river, by means of which the water level upstream of the work is raised up to the crest level—*before it can pass down the river*—" It should be high enough to ensure passing into the canal adequate quantities of water, to supply the seasonal requirements of the crops and to command all the area to be irrigated. "

5. Objects :—Anicuts are dams, built across rivers to

(i) divert water into canals or channels;

(ii) raise the water level in the river and so increase the depth, or diminish the current for navigation purposes.

6. Types of Weirs :—Weirs are classified as indicated below :—

(1) That in which water passing over the crest is dropped at once, or by steps, to a horizontal apron or on to rock.

This type is suitable when : (i) the depth of water rising over the weir is not too great, (ii) stone is abundant and cheap, and (iii) a rock foundation is available.

A wall without any apron is all that is required. Such old types are found in South India. Generally, small weirs of this type are built, but across Tambraparni and Tungabhadra rivers, weirs of magnitude have been built successfully. It is astonishing, how the then engineers succeeded in conveying the huge stones to the site and placing them in position.

(2) That in which water is lowered to the level of the river bed, by means of a curved apron.

---

* **Construction of Weir** :—India can boast that it is the only country in the world, where the art of constructing weirs across large rivers, has been extensively studied and also successfully practised. It may be said that it has the longest weir in the world and perhaps with one or two exceptions, the highest one also.

The construction of weirs of moderate height was very successfully accomplished by Indian Engineers long before English Engineers came to this country.

The Grand Anicut at Tiruchirapalli, Madras (across the river Kaveri) is a very good example of our ancient engineering work. This anicut has successfully stood the test of time (nearly 1000 years). The weir is 1080 ft. long and from 40′ – 60′ in breadth and the crest of the weir is 15′ – 18′ above the bed. About the year 1810, it is said that only some renovation was effected and then the permanent undersluices were constructed.

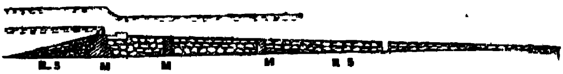
This type is adopted for surplus weirs, where the elevation has to be dropped many feet and for such positions, it is an efficient and economical form of weir. It is used where there is abundant supply of good and cheap stone, suited to masonry works and for rough stone apron.

An example of the type is the Godavari anicut. The initial cost and maintenance costs are very low in this type.

(3) That in which water runs down an inclined plane, with a uniform slope, or down a series of two or more planes, the slopes of which decrease from top to bottom.

This type is suitable where stone is cheap and abundant. It is not necessary, that this material is adopted for masonry work e. g. Anicut at Bezwada for the Krishna, and anicuts at Nellore and Sangam for the Pennar. (See Fig. 182).

OKHLA WEIR. ROUGH STONE FILLING, SUBMERGED WEIR



M—Masonry

R. S.—Rough stone.

Fig. 182.

7. Overflow type of weir :—The first and the second types noted above, come under the category of overflow dams. The principles under which these are constructed are the same as those for an ordinary gravity type of dam, and in addition, they should satisfy the following conditions :—

(1) The crest or the top of the dam should be strong enough to absorb the shock of all floating materials including debris and logs.

(2) The crest and the rear slope should withstand the force of the overflowing water.

(3) The rear toe should be protected from scour due to the high velocity it has to withstand.

8. Theoretical Profile :—(See Fig. 183). The theoretical profile of an overflow dam is a trapezium and the top width $= a = H / \sqrt{s}$, and the bottom width $b = h / \sqrt{s}$,

CREST WALL DESIGN

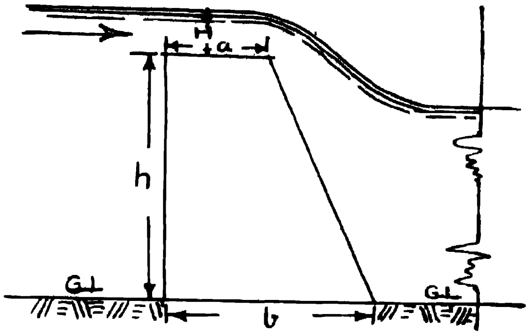


Fig. 183.

where H is the height of the water above the crest of the dam,

h is the height of the water from the bed of the river up to the crest of the dam.

s is the specific gravity of the material comprising the dam.

If the height of the dam is more than 30 ft., it is modified as noted below :—

The top surface and the rear slope are modified to accommodate the profile of the falling sheet of water. The free overfall curve of the water is generally parabolic. Beyond the *nappe*, the usual batter corresponding to the height is allowed for the rear slope, and at the toe, the form is given a bucket shape, to deflect the falling water to a horizontal direction and thus avoid damage at the toe.

Bucket :—A Bucket or Fillet is adopted in all spillway dams except when

(1) The dam is low.

(2) The discharge is very small.

(3) The foundation is rocky.

The bucket prevents the impact of the falling water from scouring the foundation at the toe of the dam, as it deflects the sheet of water to a horizontal direction,

Design :—The bucket should be tangential to the foundation. If there is a sudden enlargement where it joins the foundation, it

causes an eddy (due to the high velocity of the jet) which will erode the soft foundation, See Fig. 184.

BELOW OVERFALL WEIR

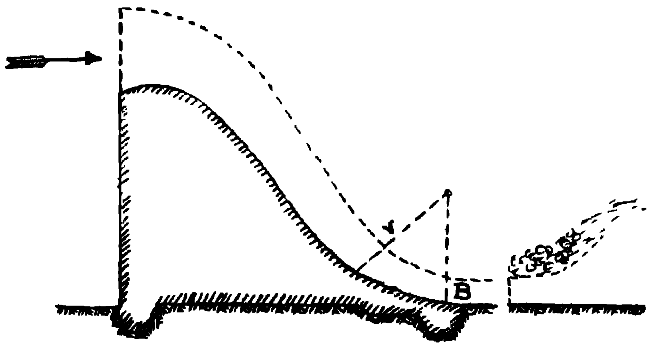


Fig. 184.

The radius of the bucket is taken as noted below. The results are approximate.

Height of dam above rock	Head on Crest		
	2	5	10 Feet
10	5	7	12
20	7	10	15
50	8	15	22
100	9	22	32
150	9	24	38
200	10	25	44

N. B.—While designing for the stability of a dam, the bucket is not taken into consideration.

9. Forces acting on an overflow dam. These are, as stated in the Chapter on Gravity Dams, the same as those for an ordinary gravity Dam and in addition, the forces are

(1) The hydrostatic pressure on the top of the dam.

(2) Impact of the overflowing water, such as logs, etc.

(3) Pressure of water flowing over the rear slope. (This is generally negligible.)

10. Stability of an overflow dam : For the stability of an overflow dam, the following points require consideration :—

(1) There should be no tension in any part of the masonry.

(2) There should be no sliding.

(3) The stress at any point should be well within the safe working limit.

(4) The top or the crest should be able to resist the shock from the waves, logs, etc.

(5) The downstream face should be protected from the formation of a vacuum.

(5) The downstream toe should be protected from scour.

11. Section of the Weir Wall-Practical :—(1) The top width should not be less than 2·5 ft.

(2) Where weir-crest shutters are used, the top crest width will have to be increased to permit of their being worked.

(3) The upstream face of the wall should be vertical and the downstream face should have a batter of 1 in 4.

CREST WALL; REAR EDGE CORBELLED

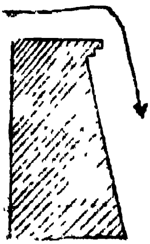


Fig. 185.

(4) The downstream face of the weir, with deep foundations, should have the batter continued as a masonry facing, to protect the concrete base.

(5) The crest should always be level in cross section; the downstream edge of the crest should be corbelled, so that the floods passing over it may be made to discharge clear of the downstream batter of the weir. (See Fig. 185).

(6) The mean width of the weirs, built of heavy masonry, up to a height of 10 ft. may be taken in ordinary cases to be equal to 2/3 of their height. For weirs of greater heights, with

great depths of discharge, their sections should be determined by means of stability diagrams, etc.

In an overflow dam, the rear or the downstream face of the weir wall should conform to the falling water.

Two sections, to match with the falling water, one with face vertical and the other with crest inclined at 45° are given in Figs. 186–187 respectively.

The shapes of the top of the crest is not of much significance. The two shapes generally adopted are given in Fig. 188.

STANDARD SECTION OF WEIR WALL

UPSTREAM FACE VERTICAL

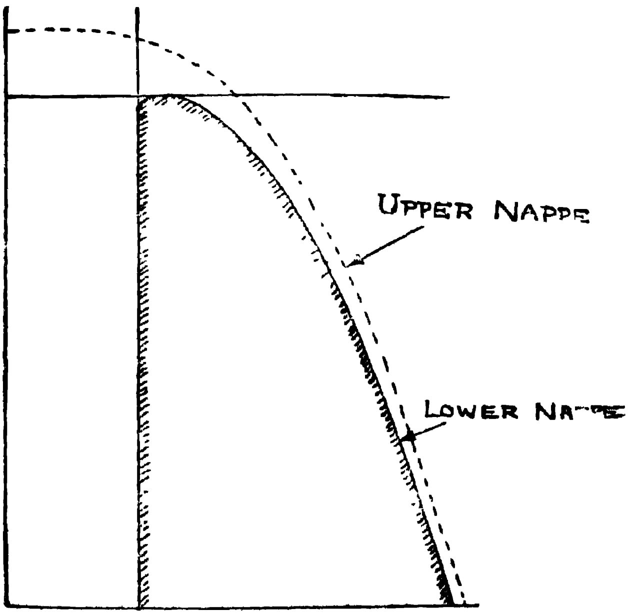


Fig. 186.

STANDARD SECTION OF WEIR WALL
FACE INCLINED 45°

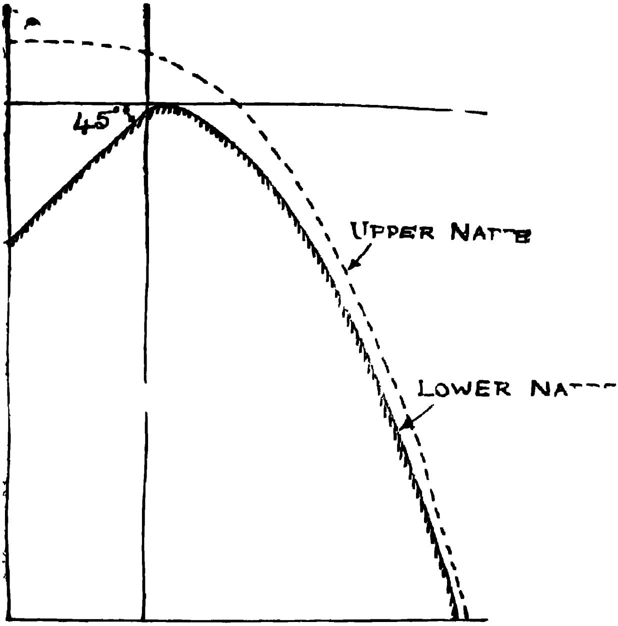
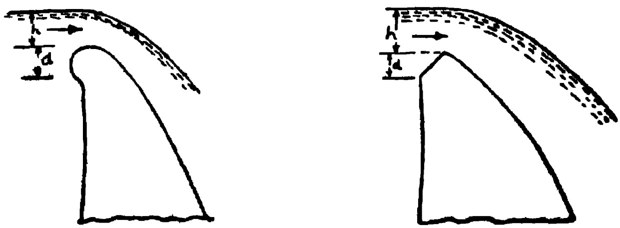


Fig. 187.

SHAPES OF THE CREST OF WEIR



$$d = \frac{1}{2}\,(h + h_a)$$

WHERE h_a IS THE HEAD DUE TO VELOCITY OF APPROACH

Fig. 188.

12. Component Parts of a Weir:—(See Fig. 189). The component parts of a weir usually are

LAYOUT SKETCH PLAN OF A WEIR

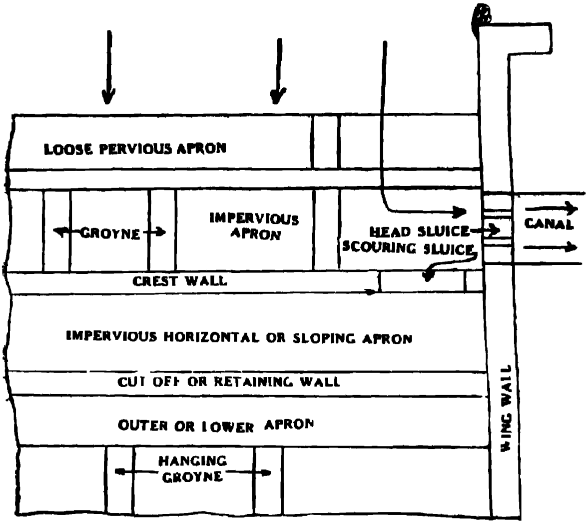


Fig. 189.

(1) the crest or body wall to retain water at the desired level.

(2) the horizontal or sloping apron of masonry, concrete, or uncemented stone with a retaining wall at the lower edge. (In the case of rough stone aprons of considerable widths, intermediate walls–bind walls parallel to the crest wall–usually horizontal or nearly so, behind the retaining wall, are provided).

(3) An outer or lower apron, usually horizontal or nearly so, beyond the retaining wall.

(4) An apron (with or without groynes) on the upstream side of the crest wall, to protect its foundation from scour.

(5) Hanging or trailing groynes projecting from and at right angles to the outer edge of the lower or outer apron, to direct the course of the water downstream.

(6) It is also usual to provide a set of under-sluices, to prevent the silting up of river bed, at the head of the off-take canal.

(7) Wing walls to connect the weir with river banks; this is necessary with flood banks or with channel head-sluices.

The above components of a weir are described in the following paras :—

13. Crest Wall :—The crest wall is to be securely founded on rock, clay or sand. If on rock or clay, there should be no leakage at all; and if on sand, there should be no material leakage near the level of the surface of the river bed.

Design :—The wall should withstand the action and impact of water. The thickness at the top of the wall should be sufficient to prevent the coping being easily displaced by floating logs or debris. The thickness is usually determined by the following formula :—

$$a = \text{thickness} = \sqrt{H} + \sqrt{d}$$

where H is the height of the dam and d is the depth of discharge over the dam.

A batter of 1 in 6 to 1 in 4 is given to the rear face, and thus the bottom thickness is arrived at. The mean thickness should not be less than 1/3 the height. When the crest wall is high, it is advantageous to strengthen it, by offset on the front face, and to reduce the batter on the rear face.

Crest Width of a Weir: The crest width of a weir must be fixed up, with reference to the following theoretical and practical considerations.

(*a*) *Theoretical Considerations.*

Let d = depth of water over the weir crest.

ρ = specific gravity of masonry.

M = coefficient of friction of masonry.

a = crest width.

The crest width 'a' to be safe against sliding,

$$a = \frac{d}{M\rho}$$

Assuming M to be 2/3, we have

$$a = \frac{3d}{2\rho} \text{ (minimum)}$$

The weir crest should not be less than this,

(b) *Practical considerations*:

(i) Height of the weir.

(ii) Nature of the drift carried by the stream.

(iii) The width required for the crest shutters, or for traffic of any kind.

Bligh's Emperical Formula:

$$a = \sqrt{H} + \sqrt{d}$$ where H = height of dam or weir

$$b = \frac{H + d}{\sqrt{\rho}}$$ and b = bottom width

Weir on Sandy bed:

The lay out of a weir and its components on sandy bed is as noted below. See Fig. 190.

On the upstream side of the crest wall and adjoining it, is an impervious apron with a cut-off wall, and on the downstream side also of the weir, there is an impervious pavement provided with a cut-off wall. (The exit gradient is from the upper cut-off wall to the lower cut-off wall).

LAY OUT OF WEAR

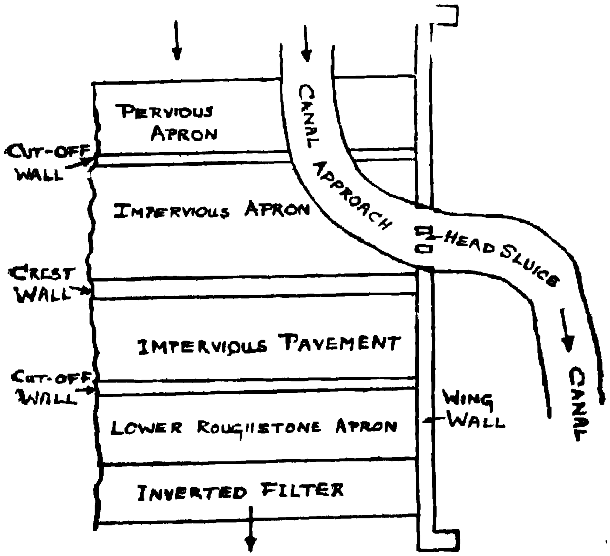


Fig. 190.

A rough stone pervious apron is provided on the upstream side of the upstream impervious apron. This apron is to keep the scour away from the weir. This apron settles due to scour and as it settles, more stone is added to form a natural slope to prevent further scour.

In the upstream impervious apron, there is no uplift pressure The lower impervious pavement checks the turbulence and the velocity of the stream.

Inverted Filter: Beyond the lower rough stone apron an *inverted filter* is provided as shown in Fig. 191. At the very

INVERTED FILTER

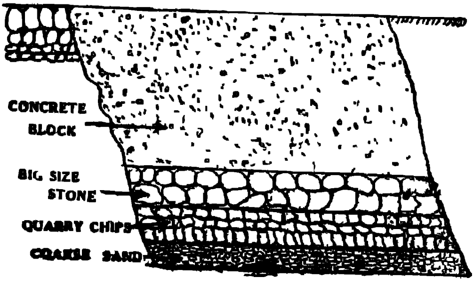


Fig. 191.

bottom, a layer of coarse sand and gravel 6 in. thick, and above it quarry chips 9 in. and then big size stone 9 in. and on the top of all, loose concrete blocks of 5 ft. × 5 ft. × 3 ft. are placed. This is to prevent any sand and small particles of soil escaping from below.

Foundation on sandy bed: If the depth of sand be not more than 12 ft., with clay or impervious soil below, it is desirable to put in concrete foundation of not less than 1½ ft. in thickness, below the surface of the impervious soil.

With deeper sand than 12 ft., it is economical to have well foundations. Here the crest wall will consist of three parts, namely, See Fig. 192,

(i) Wells from not less than 12 to 30 ft. below the surface of the river bed.

(ii) Masonry foundations above the wells to the bed level.

(iii) Crest wall proper.

SECTION OF WELL FOUNDATION FOR WEIR WALL

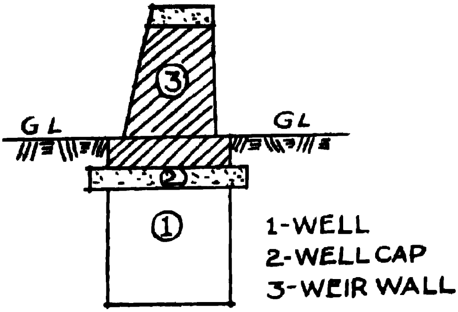
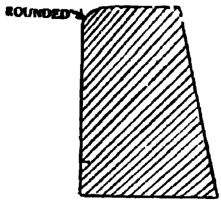


Fig. 192.

Form of Crest wall: If there is a horizontal apron, and thus a fall to the rear of the wall, the crest wall should be of the same thickness as if it were founded on rock, and it should have a batter on the rear or down-stream face, and if a sloping apron, the top width will be fixed as may be convenient for use, as a foot path, and both the faces may be vertical.

SHAPE OF CREST OF THE WEIR WALL



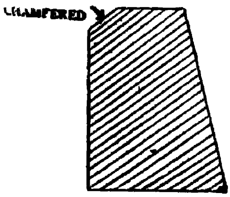


Fig. 193.

The top level of the crest wall should be somewhat higher than the F. S. L. of the channels. The exact amount of difference will depend upon the relations between the area of waterway of the vents of the head sluice and that of the supply channel.

The dimensions of the coping stones are dependent upon

(1) The character of the river.

(2) The velocity of the river.

(3) The liability of logs or trees being brought down by floods. Generally the length of the stone will be of the full width of the crest 5 ft. × 1 ft. × 8 ft. minimum dimensions, (10 c. ft. stones are not uncommon). The upstream top edge should be rounded or bevelled at an angle of 45° with not less than 4 in. sides

to the bevel to lessen the risk of trees being held up by the branches on the crest. (See Fig. 193).

14. Horizontal Masonry Apron:—The proper width of apron is dependent upon

(1) The depth of water passing over the weir before the tail water rises to crest level.

(2) The velocity of water, or the "*velocity of approach*" as it is called.

The greatest distance from the crest wall, at which water can strike the apron is got by the equation

$$x = 2\sqrt{H(d+h)}$$

where, x = the horizontal distance on the apron,
d = depth of water on the crest,
h = head due to the velocity of approach, and
H = height of the crest wall above the horizontal apron.

In practice, the width of the apron is $2x$ and is not less than $2(d+h+H)$.

SL = SLAB LAID PARALLEL TO DAM

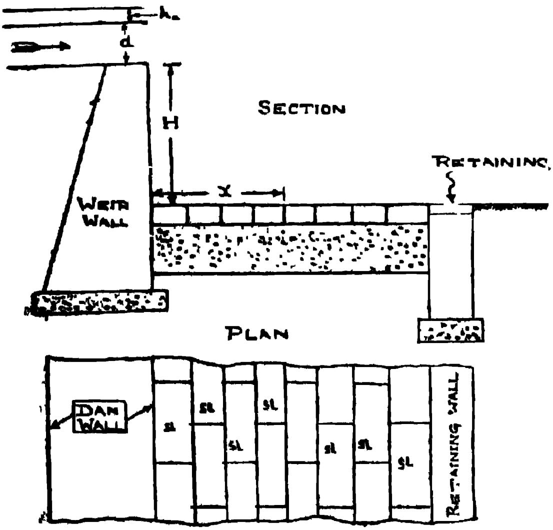


Fig. 194.

The *thickness* of this apron must be sufficient to withstand the impact of water. It is generally between 2 and $4\frac{1}{2}$ ft.

(3) Approximation to proper thickness is

$$2 + \frac{(d+h)\,H}{30}$$

The surface of the apron should be made of materials, able to withstand the action of water with sand and gravel (with which it is mixed when a river is in considerable freshes). Hard stone is the only suitable material, and it should be not less than 1 ft. thick with vertical faces dressed to make thin joints. It is best to lay the stones with their length parallel to the crest wall. Concrete is the most suitable material below the surface stone slabs of the apron. See Fig. 194.

Retaining Wall or Cut-off :—The retaining wall should be substantial and founded at a level considerably below the foundation of the crest wall in sandy beds. The top of the retaining wall should be at or somewhat below the level of the deepest portion of the bed of the river.

N. B. It is a great mistake to place it higher, as a retrogression of level in the river bed is a nearly invariable result of building a weir.

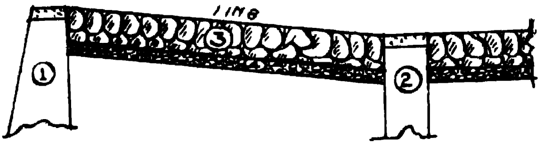
15. Sloping Apron—(1) *Of masonry or concrete* :—This should be curved and the centre from which the curve is struck should be vertically above the retaining wall, as water will then leave the apron with a horizontal motion, and with less disturbance than would be the case, were its direction inclined to the horizon. The horizontal width of such an apron varies from 3 to 6 times the vertical fall from the top of crest to that of the retaining wall. The material used is either stone or concrete with cement. Sloping aprons are not subjected to the impact of falling water and therefore may be less in thickness than the horizontal apron. $x = 3$ to 6 H.

(2) *Of rough stones* : The usual gradient of an apron of uncemented stone is 1 in 8 or 1 in 9. Thickness usually adopted is $4\frac{1}{2}$ ft. (3 ft. of quarry rubbish and $1\frac{1}{2}$ ft. of the top course with good long stones). See Fig. 195.

Percolation.—If $1/c$ is the steepest slope that the sand can have without giving rise to percolation, then $\frac{1}{c} = \frac{h}{l}$ which is the hydraulic gradient. $\therefore$ $l = ch$.

(i) For light sand and mud $c = 18$
(ii) Fine micaceous sand $c = 15$
(iii) Coarse grained sand $c = 12$
(iv) Boulders and shingle $c = 5$ to 9
l decreases from (i) to (iv)

SLOPING–APRON OF WEIR



1. Body Wall. 2 Toe or Retaining Wall 3. Apron Top Layer of Header Stones. 4 Bottom Layer of Quarry Rubbish.
Fig. 195.

To increase the length of percolation the weir should be supplemented by an extension. As it is not always economical to continue the weir itself, the apron is continued on both sides, or piles are driven (on both sides).

16. Design of Weir Aprons.—For the design of aprons, the percolation constant of the sand should be first determind The percolation constant ranges from 4 to 30 for different kinds of sand.

In South India it is taken as 12, that is, 12 horizontal to 1 vertical, and in North India 15 for Himalayan rivers.

The total length of the upstream and downstream apron must be equal to $C \times H_{max}$, where H_{max} is the water level during floods, and C is the percolation constant of sand.

Creep Theory of Bligh for the Design of Weirs on Permeable Foundations:

According to Bligh, creep is more important than percolation through the soil below it. On account of the difference in water

level on the downstream and upstream sides, the subsoil water creeps along the base profile of the weir at its junction with the subsoil and loses the head on its way downstream. The loss of the pressure along the line of flow is proportional to the length of the path traversed, irrespective of whether the path is laid along horizontal, vertical or inclined line. The measure of the reliability of the structure depends on this length of the path, and this measure is found to be different (for various grades of soil).

The subsoil water, while creeping, loses its energy and comes out at the downstream end of the downstream impervious apron with a velocity called the "Exit Velocity". For this Exit Velocity to be safe, the length of creep should not be less than CH.

$$\text{Minimum length of creep} = l = CH$$

where, H is the head (i. e., the difference between the downstream and upstream water levels), and C is the creep coefficient (or classification number) of the foundation soil.

An idea of the safe gradient may be obtained from Bligh's creep coefficient. It varies inversely with safe overall percolation gradient for that soil. That is, 1/C is the deepest slope of the percolation gradient which ensures safety against "creep" in any particular soil. Thus a coefficient of 10 means that the safe overall percolation gradient is 1 in 10.

Bligh has given the following values for C, depending upon the nature of the soil in the river bed.

Class of Soil	Value of C
1. Light sand and Mud	18
2. Fine micaceous sand	15
3. Coarse sand	12
4. Boulder or shingle and gravel and sand mixed	9 to 5

It will be seen that in spite of the higher percolation velocity through coarse grained material, the greater the size of the particles, the steeper will be the gradient.

The creep length includes cut-offs also, the length of the creep along each cut off being 2 times its depth. Bligh did not consider the vertical cut-off at the downstream end of pavement—a structural necessity—and such a cut-off is very essential according to modern theory. Hence even with Bligh's design of pavement, provision should be made for some cut-off at the end of the downstream pavement.

If the foundation soil is not to be disturbed (i. e. to ensure safety from undermining) the total creep length should not be less than CH.

Length of floors and aprons according to Bligh: Bligh has given a series of empirical formulae for obtaining suitable lengths and thicknesses of different parts of the aprons of weirs founded on sandy soil. These are (based on observations and successful structures in existence). Vide Fig. 196.

SECTION OF WEIR ON SANDY BED

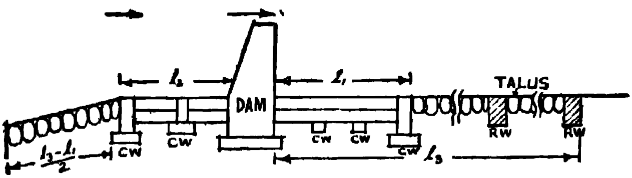


Fig. 196.

CW = Cut-off wall

RW = Retaining wall

Let l = Total length of both the upstream and downstream impervious aprons.

l_1 = Length of downstream impervious apron

l_2 = Length of upstream impervious apron

l_3 = Total length of downstream protection, (impervious and pervious.)

H = Difference of level between up-stream and downstream water levels.

H_s = Height of top of crest shutters above the lowest water level.

H_b = Height of permanent weir crest above the lowest water level.

h = Head lost in creep up to the point where the thickness is to be determined.

q = Maximum discharge in cusecs per foot-run of weir crest.

C = Coefficient of creep of the bed sand.

Then (1) length of the impervious apron downstream from the body wall,

$$l_1 = 4C\sqrt{\frac{H_b}{10}}$$

The thickness of the main impervious apron should not be less than 4 ft. It should be paved with hard surfacing material (cement or surki concrete)

(2) Knowing the value of the length of creep, the length of the upstream impervious apron is

$$l_2 = l - l_1.$$

This apron is usually constructed with a layer of masonry 1½ to 2 ft. thick well grouted with mortar, overlying a thick layer of puddle clay (2 to 2½ ft. in thickness).

(3) Total length of the apron downstream of the body wall (for vertical drop weirs).

$$l_3 = 10C\,\frac{H_b \times q}{750}.$$

For sloping and rock filled weirs,

$$l_3 = 11C\,\frac{H_b \times q}{750}.$$

l_3 is the total length of the downstream impervious apron and the downstream talus. Hence the length of the downstream talus

$$= l_3 - l_1.$$

If the talus is of greater width than 30 ft. it must be protected by longitudinal masonry walls built at 15 to 20 ft. apart. The rougher the top surface of an apron the better, as then the velocity with which the water will pass to the outer apron beyond the retaining wall will be lower and the action of the current on the outer apron and beyond it will be reduced to a minimum. The gradient of the upper part may be steeper and of the lower part flatter. (But, of course not steeper than 1 in 6).

Bligh does not give the length of the upstream protection. It may be taken as $\frac{(l_3 - l_1)}{2}$.

For *scouring sluices*, Bligh has given the following empirical formulae

(a) Length of upstream impervious pavement

$$l_2 = 7C\sqrt{\frac{H_S}{13}}$$

(b) Length of impervious and pervious downstream apron

$$l_3 = 15C\sqrt{\frac{H_b X_q}{750}}$$

The last formula was modified by W. M. Ellis who changed $H_b/10$ into $H_S/13$ since H_b does not exist for scouring sluices.

$$\text{i. e. } l_3 = 15C\sqrt{\frac{H_S \times q}{13 \times 75}}$$

The thickness of the downstream apron depends upon the actual pressure below it, according to the creep gradient line. It is given by

$$t = \frac{4}{3} \times \frac{H - h}{\rho - 1}$$

where t is the thickness of the floor in ft. and H is the total head. 4/3 represents the factor of safety.

The thickness of the downstream pavement will vary being at least 4 ft. at the downstream end against dynamic action, and it increases towards the crest wall to withstand uplift pressures.

The thickness of the upstream pucca pavement is taken as 2 ft. in all cases and uniform throughout.

Khosla's Theory for the Design of Weirs :—In 1926-29, Shree Khosla and others investigated into the troubles at the Syphon under the Upper Chenab canal (Punjab) and the following were the chief conclusions arrived at.

(1) Subsoil water flow through permeable soil is in stream lines and hence susceptible to mathematical treatment. The creeping of water along the bottom of the impervious floor can be prevented by providing deep vertical cut-offs.

(2) The foundation soil particles are disturbed by the vertical component of a force which acts at each point along the path of

flow, in a direction tangential to the path of flow of the subsoil water, from the upstream end to the downstream end of the impervious floor. This disturbing effect is the greatest at the downstream end of the downstream impervious apron where the subsoil water comes up, i. e. at the exit of the subsoil water. The disturbing force is proportional to the pressure gradient at that point. At the exit, this force is obviously upwards, and if it is more than the submerged weight of the soil particles, it will tend to lift up the soil particles. Hence at the exit point, the submerged weight of the soil particles should exceed the upward disturbing force and for this purpose, there should be a safe exit gradient.

If the exit gradient (i. e. the gradient of the subsoil flow when it comes out from under the pavement at the downstream end) is steeper than this safe gradient, the pavement will be undermined due to sand being carried away by the flow, first from the downstream end of the pavement and then from the pavement itself, thus causing progressive degradation of the soil resulting in cavities and ultimate failure.

The ratio of uplift at any point along the base of a weir on permeable foundation to the total head is independent of (i) upstream and downstream water levels, (ii) nature of the subsoil, so long as it is homogeneous, and (iii) temperature, so long as it is uniform throughout the subsoil.

Let b = length of pavement,
d = length of downstream end sheet pile,
H = Head causing the subsoil flow.

Then the exit gradient G_E is given by

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$; and $\alpha = b/d$

When the upward disturbing force at the exit is just equal to the submerged weight of the soil particles at the exit, the exit gradient is said to be *critical*. In practice a factor of safety varying from 4 to 7 is allowed in fixing the value of the exit gradient. The safe exit gradient will be thus 1/4 to 1/7 of the critical gradient. Safe exit gradients for some subsoils are given below :—

Nature of Subsoil	Factor of Safety	Safe Exit Gradient
Shingle	4 to 5	0·250 to 0·200
Coarse sand	5 to 5	0·200 to 0·167
Fine sand	6 to 7	0·167 to 0·143

(3) The undermining of the subsoil starts from the tail end of the downstream impervious floor, and if unchecked, continues upwards towards the weir wall. Undermining starts at the surface at the exit end, due to the force of seepage water being greater than the restraining forces of the subsoil which are holding the latter in position. Once the surface particles are disturbed, the resistance against upward pressure will go on decreasing, and progressive disruption of the subsoil occurs, resulting in cavities below the floor. It is absolutely essential to have deep vertical cut-off walls at the downstream end of the downstream impervious floor to prevent undermining. The safe exit gradient and the maximum depth of scour determine the depth of the cut-off to be provided. The exit gradient to be actually provided is usually obtained from the formula already given above.

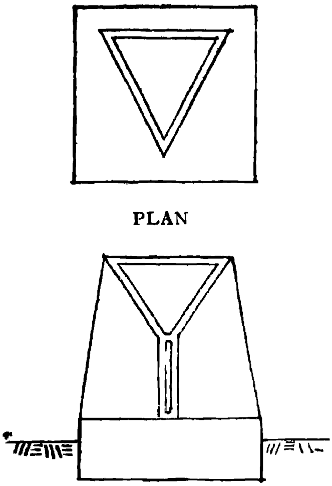
(4) Failure of weirs on permeable soil can occur either due to the sub-surface flow or due to the surface flow. Failure due to the sub-surface flow, which causes uplift on the downstream impervious floor, is most common, and so the downstream floor should be designed to withstand the uplift due to the sub-surface flow or surface flow.

Scour is caused by surface flow. If a hydraulic jump is formed on the downstream side of the weir, the surface flow also causes uplift on the downstream floor. The maximum uplift due to the surface flow should be found out by investigations.

Sometimes, friction blocks, staggered blocks and dentated sills projecting from the top of the downstream impervious floor, are provided to dissipate the surplus energy of water on the downstream side of the weir. (See Fig. 197-198-199)

A vertical cut-off is to be provided at the upstream end of the upstream impervious floor, and a few cut-offs between this verti-

FRICTION BLOCK



PLAN

ELEVATION

Fig. 197.

STAGGERED BLOCK

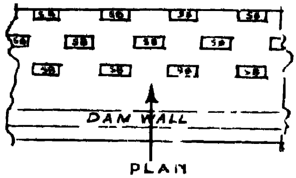
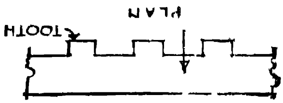


Fig. 198.



DENTATED–SILL

Fig. 199.

cal cut–off at the upstream end and the cut–off at the downstream end of the downstream impervious floor. Vide Fig. 196.

In some cases, baffle piers 5′ × 5′ spaced 5 ft. apart are constructed on the downstream side, and at the end of the apron a continuous sill wall is also constructed to dissipate the energy of the falling water. (See Fig. 200.)

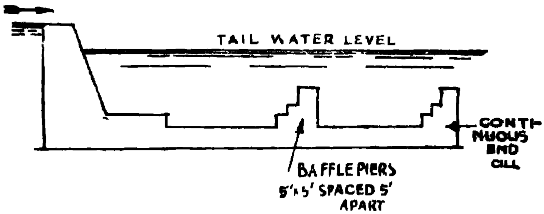


Fig. 200.

17. Outer Rough Stone Apron :—The maximum width is taken as

$2\ (d + h + H)$ for weirs with vertical fall ;

$2\frac{1}{2}\ (d + h + H)$ for rough stone apron ;

$3\ (d + h + H)$ for sloping masonry apron.

Thickness varies from 3 to 6 ft.

The surface stones near the retaining wall for 1/3 its width should not be less than 4 c. ft.

18. Aprons above Crest Wall:—These are necessary to protect the crest wall on the upstream side from scouring, along the face. These will be in the form of banks of stone with 1 in 5 slope and 3 ft. to 5 ft. in thickness, at the junction with the crest wall.

These serve also as a protection against currents induced by the action of under-sluices. When there is a heavy river current, and where the line of river is not at right angles to the anicut, more detailed protection is necessary.

19. Groynes: (1) *Above the anicut*:—Great care should be taken to see that the groynes do not breach, and no holes are formed on the off-side. They should be provided with small aprons, as they act like small weirs on the sides, against which the current sets, and with the much wider aprons at the nose. On the off-side, they should be kept as low as possible, See Fig. 201 and their action be carefully watched.

GROYNE ABOVE ANICUT

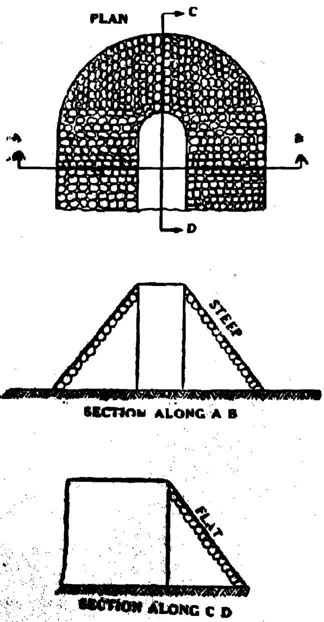


Fig. 201.

(2) *At the upstream outer wings of under-sluices*:—Here groynes are necessary, especially when the bed of the river is sandy. They should be of considerable length (twice the length of the set of sluices from abutment to abutment). The current will be rapid and so the groynes should be substantial, and the aprons wide and about 3 ft. thick. The groynes should be founded at least 3 ft. below the under-sluices. (See Fig. 202).

Hanging or Tailing Groynes: When water from one set of under sluices, or the discharge over one part of the anicut has a tendency to run along the toe of the outer apron, and to form a more or less deep channel, which might be injurious to the stability of the apron, hanging or tailing groynes are used. Vide Fig. 189.

Whenever the direction of the current of the river, after leaving the apron, is to be controlled, hanging groynes, at right

SKETCH SHOWING
SCOURING SLUICE AND DIVIDE WALL OR GROYNE

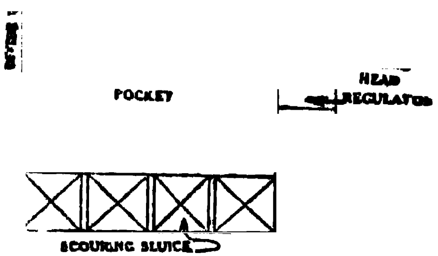


Fig. 202.

angles to the axis of the weir are constructed to divert the flood water to the desired direction. This is done by simply forming a bank of stones, on the desired line to a height of 3 to 4 ft. above the level of the apron. Rough stone is the best, but when this is not available, concrete blocks are used. (Stone is preferred, as its weight is 160 lbs. against 120 for concrete).

General .— These groynes are very useful in preventing the scouring action in the case of the river weirs. These start from both the ends of the banks. In the case of weirs standing on sandy bed, the sand should be made compact, and scouring action prevented. The upstream of the weir is subjected to the action of currents. These currents coming at right angles to the weir, dash against the weir and are converted into cross currents parallel to

the weir body wall, and these become agents of scour. (Especially the currents that run parallel to the length of the weir body-wall). Hence masonry walls are built, beginning from the weir at right angles to it, and running towards upstream. These walls or hanging groynes check the cross currents and prevent scour. Their crest level is kept about 3 ft. below the crest level of the weir.

20. Canal: Details are given in the Chapter on Channels

HEAD REGULATOR OR HEAD SLUICE:

21. Site for the head regulator: Generally the site for the head regulator is fixed at right angles to the line of the weir. Sometimes the head regulator is situated away from the weir, to get a free flow into it for the proper alignment of the channel. If the head regulator is situated away from the weir proper, a scouring sluice should be situated, just above the head regulator, to pass off all surplus water back to the river.

22. Object of the head regulator: The objects of a canal head regulator are

(1) To regulate the supply into the channel.
(2) To shut out the river floods from entering the channel
and (3) To control the amount of silt into the canal.

23. Requirements of a head regulator: The requirements are

(1) Only the required quantity should be allowed.

(2) Water entering the canal should be free from silt, hence the sill of the head regulator should be placed a little above the bed

(3) The water should be drawn only from clear levels, and as such the shutters should be arranged in tiers.

(4) The bed silt should not be allowed to enter the canal, but silt in suspension is valuable and may be allowed to flow.

24. Design of a head regulator: To design the head regulator

(a) The level of the H. F. L. in the river and the bed of the channel at the rear, should be known.

(b) The full supply discharge of the canal should be fixed beforehand.

(c) The nature and the quantity of silt should be known.

(d) Water stands to H. F. L. on one side of the head regulator and the other side is empty (when there is no water in the canal), and so the head regulator should be designed to suit this condition. Also,

(e) The area of the vent should be made big enough so that the velocity may not be more than 2 to 3 ft./sec. This is to prevent the silt from entering into the canal.

25. Construction of a head regulator: It should be founded on rock and be of masonry, of gravity design, heavy enough to resit any tendency to shear, slide or overturn. It should be flanked with ample wing walls, and have deep cut-off walls to prevent the seepage around or under the structure.

A head sluice should have ample waterway, so as to allow the irrigation water being drawn at low velocities from as high a level as possible. The head sluice should be so located, as to exclude as far as possible, the river bed silt from being carried into the canal, along with the irrigation water. For this purpose, a diversion dam is necessary to form a settling basin and to furnish head for sluicing it out, when needed.

The method usually employed for separating the silt is the *Still Pond System.* In this method, the supply required for the canal enters the pocket; the velocity of water in the

SKETCH SHOWING SECTION AT HEAD SLUICE WITH TIERS FOR SHUTTERS.

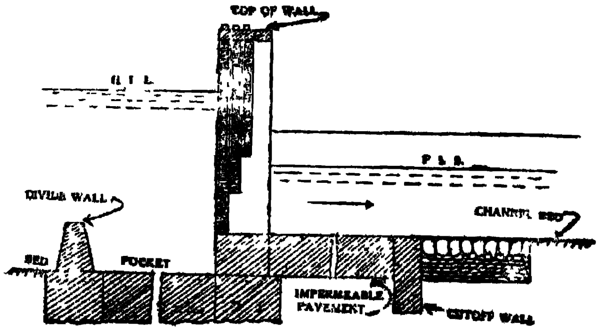


Fig. 203.

pocket is very much reduced on account of its excessive water-way and the silt deposits in the pocket and only clear water enters the canal.

The head regulator should be provided with 2 or 3 tiers or rising sill gates. The object of this is to force into the canal, only the surface water, which is free from coarse silt–See Fig. 203.

The length of the apron provided on the downstream side of the head regulator should be properly calculated. The uplift pressure here is more than that for the weir proper, but the velocity of the retreat is small, and to be on the safe side, the same length as that provided for the weir is allowed for the head regulator also The total length of the apron both upstream and downstream is 2H to 3H if c is 12, and it is 4H to 5H if c is 15, and the length of the ordinary previous apron downstream is made from 2D to 5D, where D is the depth of full supply in the channel.

The flanks on either side of the head regulator should be above the H. F. L., say 3–4 ft. above it. Sometimes a bellmouth is given for the entrance to the head regulator with 1 in 10 divergence to allow for more discharge.

As the head wall is situated just near the scouring sluice, the draw of water from the scouring sluice would affect the foundation of the head regulator. It should be properly attended to, and for this purpose a good impermeable apron with a curtain wall should be built in front.

Scouring sluice or Under–sluice

26. Objects of the scouring sluice .—The objects are

(1) To scour the silt deposited on the river bed, upstream of the canal regulator, and control its entry into the canal.

(2) To preserve a clear and defined river channel, approaching the regulator.

(3) To lower the highest flood level.

27. Location of scouring sluices :—Scouring sluices are made in the crest wall of the anicut. As the object of the scouring sluice is to prevent the formation of the approach to the head [illegible] scouring sluice is as close to the

The best location for the scouring sluice is near the flank of the river * and not the centre (of the river). If it is located in the centre, the disadvantage is that one cannot go there to operate, and secondly, silt should not accumulate near the head of the canal, and hence the location near the flanks. For this purpose, an approach channel should be excavated to lead the river water to the head of the canal and necessary protective works should also be provided here.

28. Function of scouring sluice :—The main functions of any scouring sluice are

(1) To remove the silt from the bed of the river at the head sluice where it is situated.

(2) Training of the flood, especially during the beginning of the monsoons.

(3) Minor regulation of water without using the weir shutters.

29. Level of a scouring sluice :—The level of the sill of the scouring sluice is usually kept at the lowest bed level of the river. This level should be 2 or 3 ft. or even more below the bed of the channel at the beginning. The level of the flooring downstream side of the scouring sluice should be lower than the sill. It may even be taken as the lowest retrograded level (likely retrogression) and a series of drops should be provided for the purpose.

30. Action of scouring :—The scouring action in the sluice is due to

(1) The silt laden flood.

(2) The action of the gate.

(3) The depth of water falling from above.

31. Gates for scouring sluices :—For scouring sluices, it is an advantage to provide them with two sets of gates. They are :—

(1) The Regular gates.

(2) Temporary gates (in the form of planks to be placed in the grooves of piers provided for the purpose). The advantage

* If there are 2 canals one in each flank, two scouring sluices will have to be constructed.

of the system is, it is useful during the repairs of the gate and it also acts as a safety measure.

In the old types of scouring sluices, the gates were only 6 to 8 ft. in width. Now, after the advent of good gates with free rollers, etc., the breadth is increased even to 30 — 50 ft.

N. B.:—It is an advantage to have a gate broad, in that the logs, as they are received from the flood, will not be obstructed by the narrowness of the gate, while they pass down through them, and for this purpose, every alternate pier is made to project to guide the logs into them.

The power to lift the shutters is by turbines fixed on the flanks.

32. Construction of scouring sluice:— The discharging capacity* of the scouring sluice should be large in comparison to that of the approach channel so that the silt may readily be removed along it, and the sluices should be capable of being operated under all conditions of river flow. To achieve this object, a bridge downstream of the scouring sluice, should always be provided to lift the gate, etc.

The foundation must be made strong enough to withstand the intensity of discharge. This requires special care, as the velocity of water passing through them is very considerable. Moreover, when the high velocity current through the sluice meets the less rapid current, passing over the anicut, whirlpools and scour holes (sometimes to great depths) are formed.

N. B.:—Scouring sluices are also constructed in channels at important points to discharge all the water, when it becomes necessary to quickly empty the canal in case of a breach, or threatened breach, during abnormal rains.

For large canals a permanent weir with a scouring sluice is built immediately below the headwork, and to the downstream side of it (weir with scouring sluice) a regulator is provided across the canal. The object of the scouring sluice is to scour away all the silt at the main reach of the canal, and to discharge the surplus water that may enter the canal and also for any repairs necessary to be done to the channel lower down.

* The discharging capacity is sometimes taken as twice that of head sluice or 10 to 15 p. c. of the maximum discharge of the river.

Scours are generally formed at the rear of the scouring sluice (as stated above) by the falling flood and also by the low cold weather supply (and not by the maximum discharge); because during this season there is no back-water, and the clear water has always a greater scouring action. Beyond the scouring sluice, on the opposite side of the head regulator, a dividing wall is constructed to isolate the general flood water from entering into the canal (See Fig. 204) This divide wall prevents the currents parallel to the weir and trains and directs the course of the river. The length of the dividing wall is generally designed to be equal to the width of the river between the bank and the dividing wall. The

LAYOUT OF SCOURING SLUICE AND HEAD REGULATOR

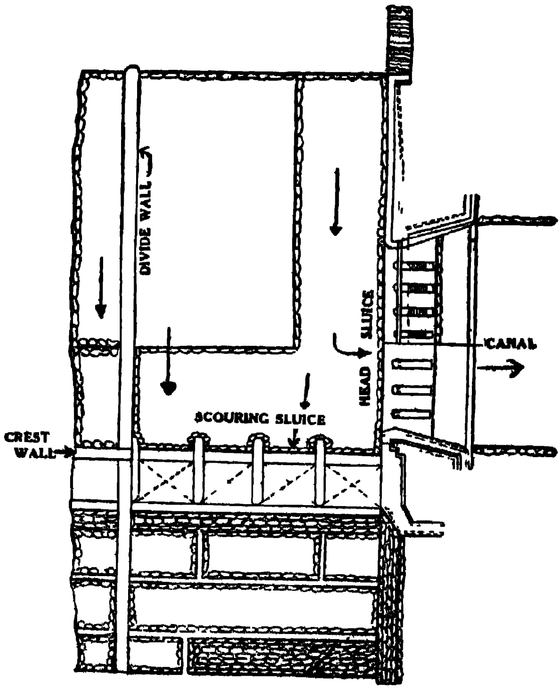


Fig. 204.

depth of foundation of the divide wall is taken to be 1/3 the height of water or the depth of water at the place.

Cross currents in a river are produced when it (the river) receives its supply from different directions, and for this purpose groynes are constructed to prevent these cross currents.

The wing wall for the bank of the river downstream side beyond the head regulator should be parallel to the river (See Fig. 164) and not splayed, as, in this case, the bank will get scoured.

As seen in Fig. 164, a pocket is formed between the main head regulator and the divide wall. This confines the effect of the scouring sluice and scours a deeper channel against the head regulator, which helps in preventing silt in the pocket from entering the canal.

Advantages of scouring sluices :—Scouring sluices will draw silt from near the head sluice and keep the river bed in front of it scoured out for a considerable depth below the sill level of the head sluice, and so minimise the amount of bed silt passing into the channel.

33. Design of scouring sluice :—The thickness and the length of flooring should be designed as suited to the main weir, but the flooring and *talus* downstream should be stronger than those of the weir. The width of the floor of the scouring sluice is given by the empirical formula.

$$(\text{Width})\ W = 7\,c\sqrt{\frac{H}{13}}$$

and L the length of the total width of the aprons below the shutters, by the formula L =

$$15\,c \times \sqrt{\frac{H_b}{13}} \times \sqrt{\frac{q}{75}}$$

where H is the difference between the top of the crest shutter and the floor of the scouring sluice, q is the discharge per running ft. in cusecs, and H_b = height of permanent weir crest above bed.

The depth of the talus should be about 4 to 6 ft. i. e., it is taken roughly as 1½ times the depth of the floor of the down stream apron of the weir.

34. Divide Wall: This wall is simply a long defined groyne, built between the weir proper and the under-sluices. It separates the turbulent river when in maximum floods from the pocket in front of the channel head regulator. This wall plays an important part in controlling the entry of silt into the channel by enclosing a pocket of very nearly still water, and by separating it from the alluvial river, in floods.

This may be an earthen bund protected with stone pitching on both sides. The upstream end is provided with extra strong protection in the form of an apron, but as this requires a lot of space, it is now usual to build it with solid masonry. The wall should extend to a considerable distance above the upstream vent of the head sluice.

35. River Control Work (Training Bank): This induces a clear and direct flow to the weir or anicut and it reduces the number of spurs needed along the margin. A free board of about 3 ft. should be allowed above H. F. L. for the afflux head.

A squill round the flank of weir generally entails serious risk of a deep channel being cut round the flank through which all or a large part of the river flow may be diverted. This is one of the causes of the failure of weirs, and hence the adequacy and proper maintenance of such flanks is a matter to be specially attended to.

36. River Regulator or Barrage: This is a weir with vertical lifting gates which are operated from a regulating bridge or platform above the High flood level.

(1) Barrages are inconvenient to operate; especially in floods, when they are even dangerous.

(2) Though automatic in theory, they are not so practical, and manual labour has to be employed occasionally.

(3) If all the gates are opened simultaneously, there is a heavy strain on the weir wall; and hence the barrage does the work of the weir without constricting the waterway.

In the modern barrages, standing-wave-flumes are used for dissipating energy. The main weir is constructed with slopes on both the sides; 1 in 5 upstream side, and 1 in 3 downstream side.

In place of masonry curtain walls, steel piles are now used to act as cut-off walls, as these are easier and quicker to use, and also

cheaper. Also if curtain walls are constructed, the depth is limited and piles can be taken to any depth and these are easy to handle. See Fig. 205.

SECTION OF A MODERN WEIR

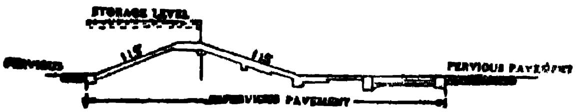


Fig. 205.

Construction · The construction of modern barrages is mostly of cement concrete, as such, the pavement becomes one monolithic mass, unlike the masonry structure which separates and blows out from uplift pressure.

On the downstream side, friction blocks are used to throw the jets of water with high velocity and reduce erosion. The design of the gates is also much reduced, the span being 60 ft. as rollers (frames bearing) are introduced to reduce friction. Below the impervious stratum, pressure pipes are introduced to observe the pressure, so that any likely danger can be known beforehand.

Silt excluders in the pockets of scouring sluice, and silt ejectors (a series) below the head regulator are all provided, the last ejectors being situated just above the cross regulator.

37. High Coefficient Weir.—These weirs are mostly constructed with lengths smaller than other types of masonry weirs. For an ordinary broad crested weir of an anicut, the formula is $Q = 3{\cdot}1\,LH^{3/2}$, where the constant C taken is to 3·1, but in a High coefficient weir, 3·1 can be made as big as 4·26, then a smaller length of weir suits the purpose.

Object of High coefficient weir :—The construction of a High coefficient weir serves the following objects ·—

(1) The length of the dam or spillway is much reduced.

(2) The depth of the spillage over the dam can be reduced. This means the area of submersion is reduced.

(3) The height of the dam can be increased, (because the depth of discharge over it is reduced). Hence greater storage can be effected.

Advantages :—As the depth of spillage is smaller, there is less danger at the rear toe of the dam, as otherwise heavy and costly protective works would have been necessitated.

Design of the weir:—This depends on the nature of the profile at the top, the slopes of the front and the rear face, and the ratio of the depth of spillage to the height of the weir measured upstream. For the profile of the High coefficient weir, several experiments were conducted at Krishnaraja Sagar, Mysore, and it was proved that the parabolic form was the best, giving a coefficient of 4·26.

To prevent or protect the scour at the rear, several methods employed are as follows :—

(1) Construction of an apron.

(2) Construction of Baffles : The best form of a baffle is Rehbock Dentated sill.

(3) Provision of the water-cushion.

(4) Provision of the hydraulic jump.

(5) Hydraulic roller foundation. At the bottom, the hydraulic roller is induced which pushes the formation of the pool towards the downstream side, and thus protects the toe. The hydraulic roller shoots the water at an angle of 45° and affords the maximum range. An upturned bucket is also provided as shown in Fig. 206 which (i) adds to the stability of the weir, (ii) helps to relieve the bed, from the impact of water.

HIGH COEFFICIENT WEIR

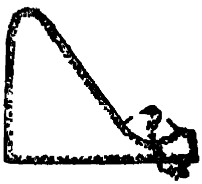


Fig. 206.

The crest width of the weir is given by the formula

$$a = \sqrt{H} \text{ and the base width } b = \frac{H + d}{\rho}.$$

38. Working conditions of Anicut and Tank weirs contrasted.

Anicut	Tank weir
(1) The most common soil met with is sand.	Foundation for tank weirs is almost on every type of soil, and rarely on sand.

Anicut	Tank weir
(2) The tail water backing-up during floods, affords to some extent protection to the downstream apron (at the toe).	Majority of tank weirs, built on soft soil, are exposed to damage, by scour at the toe of the apron and retrogression of levels.
(3) Depth of water passing over weirs is usually great, and continuous for many days in flood season.	Depth of water passing over tank weir is very small, and that too, for a few hours or days, during maximum floods only.
(4) Anicuts are worked heavily and continuously; this should be considered during design.	Tank weirs are therefore worked less heavily, and far less continuously, than river weirs.

Questions

(1) Explain, with sketches, the different types of river weirs, stating the limitations and advantages of each type.

(A. M. I. E. May 1951)

(2) Draw a sketch plan of a headwork of an irrigation canal with all its component parts and explain the function of each of them.

(A. M. I. E., May 1953)

(3) Draw a sketch plan of a head work of an irrigation canal with all its appurtenant works explaining the functions of each of them.

(A. M. I. E., November 1953)

(4) Describe with neat sketches the several component parts and their functions of a head regulator. Indicate the method of design.

What constructional devices would you suggest at the head works for the exclusion of silt from the channel?

(Mysore University, September 1950)

(5) Write a short note on : High Coefficient weir.

(Mysore University, April 1949)

(6) Write a short note on : Falling Apron.

(Poona Univ. Oct.. 1952 B. E.)

(7) What are the functions of (i) downstream apron and (ii) downstream talus in a weir on an alluvial river?

(Poona Univ. Oct. 52. B. E.)

(8) What are the functions of the head regulator on an alluvial canal?

Draw a plan, elevation and cross section of the head regulator on an alluvial canal.

(Bom. Univ., Oct. 1949, B. E.)

(9) What do you understand by "High Coefficient weirs"? Show how the glacis of a High coefficient weir may be designed.

(Bom. Univ., April 1953, B. E.)

(10) Explain with the help of a sketch the function of Reversed filter.

(Gujarat Univ. April 1945, S. E.)

(11) Distinguish between Barrage and Solid Diversion weir.

(Gujarat Univ., April 1953. S. E.)

(12) Write a short note on: Groyne wall.

(Mysore Univ. March 1948)

(13) A weir is to be built across a river running in fine micaceous sand ($c = 15$). The weir wall will be of rubble masonry 8 feet high above down water level surmounted by 3 ft. falling shutters. The weir will be of type A, with the apron at low water level. The maximum flood discharge per running foot of weir is 70 cusecs, and the maximum flood level will be 8 ft. above the crest of the weir. The crest width of the weir may be taken as 6 ft. and the bottom width as $8\frac{1}{2}$ ft. with front and rear batters equal.

(Mysore University, March 1950)

(14) Explain the function of Head Regulator and cross regulator and indicate the difference between them.

(Poona Univ. 1955)

(15) (a) Explain —

(i) Bligh's Creep Theory for the design of weirs on permeable foundations and

(ii) Khosla's method of design of modern weirs.

(b) Design and draw a dimensioned sketch of the cross section of a river weir on fine sand ($c = 15$) following Bligh's Theory.

F. S. L. of Canal	... ...	R. L. 30·00
Bed of River	... ...	R. L. 20·00
Height of drop shutters.	...	3′ — 0″

Flood discharge per foot of weir 75 cusecs.

The dry weather flow of the river is just sufficient to meet the canal requirements and the weir is just submerged when 4 feet of water is passing over the crest.

(Mys. Uni., B. E. Civil, Sept. 1954)

(16) Discuss the conditions that make a site more suitable for an anicut than for a dam.

An anicut is to be constructed across a river consisting of coarse sand ($c = 15$). The body wall having a top crest of 5 ft. and the same slope on both sides is to be constructed of mass concrete 10 ft. high above the low water level surmounted by 3 ft. falling shutters. The weir is to be provided with the main apron at the L. W. L. The maximum discharge per foot of the weir is computed at 85 cusecs and maximum depth of water passing over the crest is 8 ft. The weir becomes submerged when 4 ft. of water passes over the crest.

Design the weir, making suitable assumptions.

(Mys. Uni., B. E. Civil, Sept. 1955)

(17) A masonry weir is 20 ft. high and discharges water to maximum depth of 8 ft. above its crest. The specific gravity of masonry is $2\frac{1}{4}$. The ratio of depth of water 'd', passing over the crest to the depth of water 'D', below the weir is 0·4 as determined by calculations. Sketch the section of this weir showing all the precautionary measures you would recommend for its safety if the weir is founded on friable soil.

(Mysore Univ., B. E. Civil., April 1957)

(18) (a) Draw a neat dimensioned Sectional-elevation sketch of a Cross Regulator over a canal partly in embankment, with discharging capacity of 1200 cusecs.

(b) How is discharge regulated ?

(c) When is the Regulator required to be operated, and when closed ?

(Poona Univ. B. E. Civil. April 1957)

(19) Bring out the difference between Diversion Weir and Waste Weir.

(Poona Univ. B. E. Civil. April 1957)

(20) (a) State the main points of difference between Khosla's thoery and Bligh's creep theory.

(b) How does a barrage differ from a weir ? Draw a neat L-Section showing the bed protection provided on the downstream of a barrage.

(Gujarat Univ. April 1957)

(21) What are the different types of weirs constructed on alluvial weirs and under what conditions is each one suitable ? Give sketches for two of them.

(Gujarat Univ. April 1957)

(22) Write short notes on . Scouring sluice.

(Gujarat Univ. April 1957)

CHAPTER XIV

FLOOD AND ITS DISPOSAL

1. Floods are caused by heavy rains, of great intensity occurring at short intervals and these are influenced by causes already indicated in Chapter V where run-off is described. The intensity of flood and the periodicity also cannot be forestalled, and much less calculated. The experience of the floods of recent years 1955–56, in our country, has opened the eyes of all engineers. These floods were never anticipated. For the last more than one century, this kind of flood has never happened, and the damage that has occurred is unprecedented and enormous.

2. The disposal of flood is a very important problem, and the Union Government has appointed a committee of expert engineers, to study the details and tabulate rules and regulations for the prevention, control and disposal of the floods so that floods as occurred recently, may not recur and cause damage to the country and the people.

3. Collection of Data :—To prevent damages by floods and control them properly, data should be collected for a number of years continuously. In America, for the river Mississippi, information is being collected for more than a century, and even now final results are not achieved. The following information or data should be collected :—

(1) Rainfall statistics in the catchment area, for very many years and for good many stations.

(2) Study of the topographical features of the catchment area.

(3) Climatic conditions of the area.

(4) Maps showing the area affected at the different stages of the flood.

(5) Gauging of the river flood-discharges and checking them with the rainfall records (calculate the river cross section and slopes and the flow over weirs or through bridges, if any) and preparing a hydro-graph of the daily flow.

(6) The extent and nature of the damages caused from time to time (population affected, and amount of loss to people, Government and private companies, etc.).

River Control

Rivers are as harmful as they are useful to man. To prevent the harmful effects, they should be kept well under control.

This harm or damage is caused by the river changing its course, or by overflowing the banks. In both the cases, there is damage to the lands or loss of property or even life. Hence to prevent these dangers, action should be taken as detailed below.

(4) Course of a river : Before studying the causes and the remedies for floods, it will be well to know about the nature of the course of a river.

The river, in its course from the hills to the sea, may be divided into 5 regions.

1. Mountainous 2. Sub-mountainous, 3. Trough 4 Alluvial and 5. Deltaic

1. *Mountainous region* : Here the stream flows on a rocky bed, through narrow gorges generally, and with abrupt falls. The stream has mostly a straight course and a high velocity, and its water is clear. The bed in the higher regions is eroded and silt deposited in the lower regions.

2. *Sub-mountainous region* : In this region also, the river is still steep ; but it is wider here, with small nallas running into it.

3. *Trough region* : Here the river flows in a cutting or trough made by itself, and has a tortuous course.

4. *Alluvial region :* In this region, when the river is low, it has only one course; but in floods, the river may have more courses or channels. The river has a tortuous course here.

5. *Deltaic region*: Here the river constantly changes its course, and has a tendency to subdivide and forms sector-like or deltaic areas by its side. The river does not serve as a drainage, but on the other hand, good embankments are required to protect the country all round.

(5) Causes for the devastation by floods :

(1) If there is excessive precipitation in the catchment area, there will be abnormal overflowing of the banks, due to the abnormal discharge in the stream.

(2) Even with normal flood discharge, if there is a rise in the bed of the river, due to deposit of detritus, there will be overflow over the banks.

(3) Great variation in the flood flow. The variation may be

(a) between dry weather flow and normal flood flow.

(b) between normal flood flow and highest flood flow.

Due to this variation, the regime of the river is not properly established and hence the flood.

(4) Owing to obstruction in the bed of the river by detritus, the river may change its course, or erode the banks and thus cause damage. This causes the maximum damage.

The intensity of flood havoc and its frequency, as already mentioned, depend upon

(a) Erodability of the catchment area.

(b) Heavy rainfall in the area.

(c) Rapid change in the bed of the river. (This causes the detritus carried by the river in the steep portion to be deposited in the flat and level portion).

Remedy for the above: The only and the best remedy which is quick and effective, is construction of embankment for the river.

With regard to the construction of these embankments, special attention should be paid to the following points:

(1) The location of the embankment should be properly selected.

(2) The embankment should be constructed on both sides of the river.

(3) The cross section of the river should be sufficient to pass the maximum discharge; that is, the embankments should be located at proper distances from the edges of the river banks. These are called *Retiring Banks.*

The embankments are effective (a) where the river does not carry much coarse silt or heavy detritus (b) where there is no rapid change in the bed of the river.

Disadvantages of banks. They tend to prevent local drainage from entering the river, and as a result of this, the surrounding area will be flooded, and to remedy this, a parallel

drainage channel should be formed to the nearest valley, to lead this water.

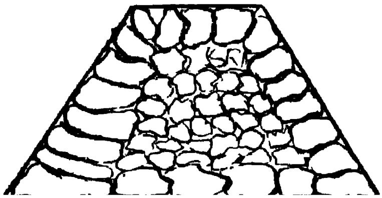
River Control: This is a very important item, especially during floods. Generally control is also necessary to lead the river water to a structure such as a weir or barrage, or a bridge or for a navigation canal. The methods or works usually adopted are described below.

I Spurs: A spur is an obstruction constructed across the course of a stream and it starts from the bank and protrudes into the river, to divert the flow.

The spurs may be (1) Impermeable (2) Permeable.

1. *Impermeable spurs*: Examples :

A solid stone construction is an impermeable one. This is generally built trapezoidal in section. See Fig. 207. This is a



Stone spur.

Fig. 207.

costly construction. An earthen bund with an outer coating or protection of stones (pitching). See Fig. 208. This is cheaper than the one above.

Spur with Earth and Protected with Stone.

Fig. 208.

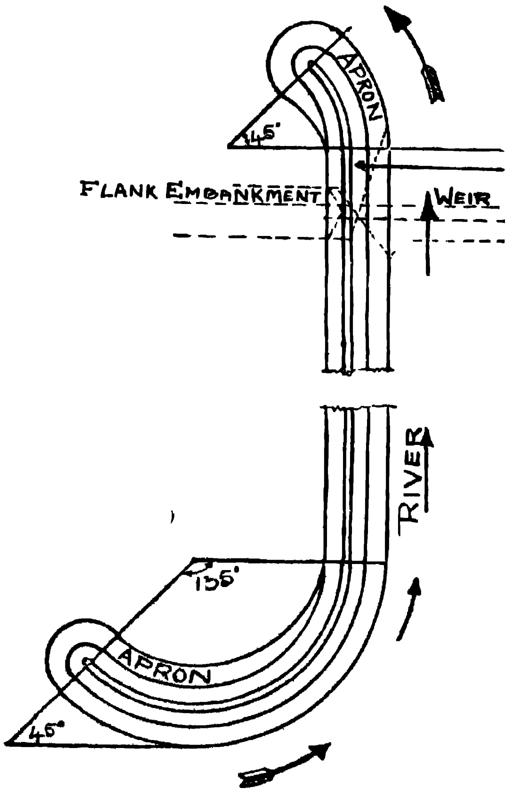
2. *Permeable spurs*: These are mostly of a temporary nature, but in some cases, these may become impermeable, due to

silting up, and may become permanent. These permeable spurs may be of brushwood, bamboos, logs of timber, etc.

According to requirements these spurs will be built either singly or in series, either parallel to the flow of the river or at an angle to it.

II Guide Banks: These are an improvement on the original spur system. A guide bank is generally an earthen bund with a protection of pitching and chips or jelly backing behind it.

These are constructed to guide (as the word indicates) or to lead the water of the stream in the proper direction, and to the



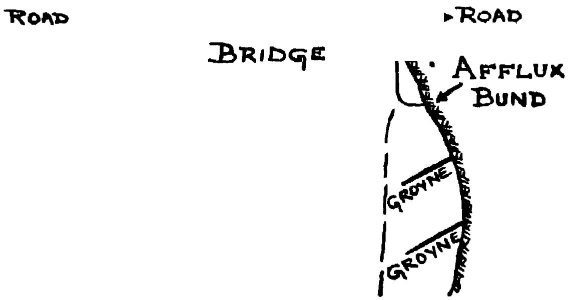
Guide Bank.
Fig. 209.

required depth without overflowing the banks. They are constructed as approaches to a bridge (Highway or Railway) or a weir.

N. B. Guide Banks were originally introduced by Mr. J. R. Bell (and are therefore called Bell banks) and later on, they were improved by Sir Francis Spring.

At the upstream side of the banks, to guide the water gently to them without the least obstruction, a curve of 135° as shown in Fig. 209 is given to the bank, and a bell mouth is also formed.

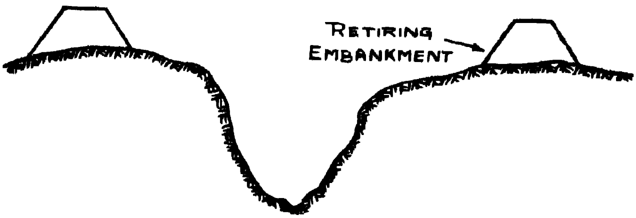
Near bridges, in addition to guide banks, afflux bunds and groynes also are constructed, as a further safety to the structure of the bridge, See Fig. 210.



Afflux Bank and Groynes
Fig. 210.

For a weir, to lead the water from the upstream side to the site of the weir, guide banks are constructed. These are also called *Dykes or Marginal Embankments*. They are constructed parallel to the river banks, and their tops are kept above the highest flood level, to protect the neighbouring lands from the effects of flood. These dykes are protected with rough stone revetment or pitching, towards the waterside, and their toes also are protected with flat pitching to prevent scours. For the banks

or dykes to be safe, they should be located beyond the meander line of the river, see Fig. 211.



Retiring Embankment
Fig 211.

Guide banks induce a direct and clear flow towards the weir There is no room for the river to swing about, and there is no possibility of any cross flow, just upstream of the weir. The length of the upstream guide bank should be longer than the length of the weir (roughly about 1/10 of the length of the weir more) On the downstream side, this is made 1/5 longer. The top level of the guide bank should be well above the HFL (about 3 feet).

Falling apron: This is mostly used in incoherent material, such as, alluvium, and not in clayey or rocky soil. The angle of repose of the material should be flatter than that of stone.

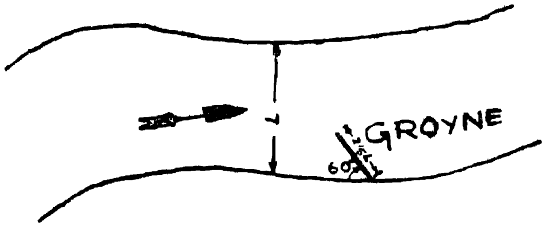
Instead of constructing revetment or pitching to the bank a (what is called) *Falling apron* is used. The coarser the material, the bigger the size of the stone used. Here the stones are simply thrown in front of the bank over the silted berm. As scour takes place in the bed of the stream, these stones fall down to the bottom and new stones are dropped in their places. This process is called *launching*. The thickness of the apron is given by the formula $t = 3/50 \times Q^{\frac{1}{3}}$.

N. B.—When the river falls down quickly, slips sometimes occur in guide banks owing to the pressure of water contained in the soil behind.

III. Groynes: A groyne may be defined as an obstruction constructed from the bank of a stream and projecting into it, and intended to divert the flow.

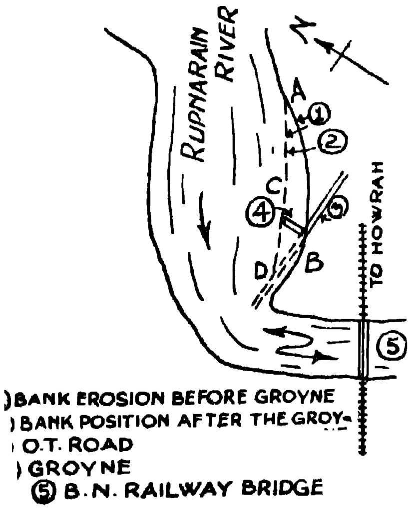
Groynes are of 2 classes (a) Attracting Groynes and Repelling groynes.

(a) *Attracting Groynes*: These are built where it is intended to attract the current to the banks, to fix the deep water channel close to it, and the river here adheres to a particular bank. The length of the groyne is taken as 2/5 of the upstream meander. See Fig. 212.



Attracting Groyne.
*Fig. 212.

Recently, the advantage of the construction of Groynes has been well experienced, as seen at the Rupnarain river works where the scoured portion near the B. N. Railway bridge has been filled up gradually. (See Fig. 213).*

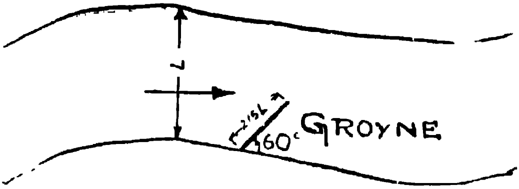


Rupnarain River near B. N. Railway B idge.
Fig. 213

* Fig. taken from the Bhagirath Magzine.

(b) *Repelling Groynes*: These are intended to repel the stream, i. e., to divert it away from the banks, and silt is thus deposited between two such groynes and it strengthens the eroded bank. The groyne is constructed at roughly 60° to the bank, and the length is made the same as for the above type; i. e. 2/5 of the meander length. See Fig. 214.

Groynes are constructed to train the river, especially where it is required to restrict the side movements of the stream. They lead the stream straight to the structure (weir or bridge). They are constructed starting from the bank and continued into the river across it at an angle which depends upon the nature of the current. They should be built at proper distances apart, and should be located at selected places, so that the diversion of the stream is properly achieved.



Repelling Groyne.
Fig. 214.

If the training of the river is for navigation purposes,

(1) It may have to be widened (specially at the bends). This may be effected by realignment, if necessary.

(2) The stream may have to be made deeper. This is effected by dredging to the required (minimum) depth; and if the depth is sometimes to be increased, weirs are built across the stream (to increase the depth).

(3) If there is a rapid in the stream, a by-pass in the shape of a lateral take-off channel constructed. See Fig. 215.

BY-PASS
When the stream is in a rapid.
Fig. 215.

Damodar Valley Corporation.

The works here are also intended to give protection from floods. Though the Damodar is a small river, yet its peak floods are out of all proportion to its size. A fertile area is always under threat; and Calcutta's communication is also in danger; and DVC gives protection against floods.

The dams are intended to moderate the floods; and even as it is today, when the works are partly completed, they have retarded the floods to a good extent, and it is believed that, when all the dams would be completed in 1958, there will be full immunity and " floods will soon be a thing of the past."

Recent Floods in India : The states that have suffered very badly during the recent unprecedented floods are Assam, Bihar, Uttar Pradesh, Orissa and Andhra.

Causes. The flood havoc in U. P., Bihar, Orissa and Assam are due to the following reasons :—

(1) The plains here are about 200 to 500 ft. above the sea level, and are right at the foot of the Himalayas (the highest mountains in the world) and the distance from the highest point to Bihar is only about 100 miles, that is, the slope of the river is very great in this short length.

(2) The mountains here are geologically of recent formation, and are liable to weathering and erosion.

(3) This area is subject to heavy rainfall (highest intensities in the world).

(4) For the above reasons, heavy and large quantities of detritus are carried down the mountains and deposited just at the beginning of the plains where the country is flat.

(5) The junctions of these torrential streams with the main rivers are just at the commencement of the plains, and, as a result, these rivers change their courses easily and cause heavy damage.

(6) Due to the deforestation in the catchment area, and to the seismic changes in the locality, the nature and frequency of floods are either changed or increased.

6. Remedies suggested : In the rapid portion of the river, check dams may be erected to prevent erosion, and generally embankments may be constructed throughout. Embankments in

the higher reaches are very costly owing to the steep slope and high velocity of the current, (good protective works are required for them). Embankments in the lower reaches are outflanked by the rivers changing their courses, even from where the embankment starts. The heavy rainfall in the locality also presents the problem of drainage to be solved: again valuable fertilising matter does not reach the river.

All these conditions will have to be studied; and the problem is not very easy of solution economically.

In the Uttar Pradesh, the floods had no parallel in recent times. The (1955) flood exceeded the heaviest on record. i. e., the flood of 1871. Both the rivers Gomati and Tonds did the havoc.

In Assam the township of Dibrugarh suffered badly both in 55 and 56. This town is proposed to be protected well, taking expert advice. Some of the protective works of last year were washed away and it is now proposed to adopt other measures and designs recommended by experts.

The eastern districts suffered very badly (about 10,000 sq. miles were inundated and there was a damage of nearly 4·5 crores).

For flood fighting, a project for 25 crores is sanctioned and about 4·5 crores work is in operation to tame the rivers Gagra and Rapti which are the tributaries of the Ganga and Yamuna.

A marginal bund along the Ganga from Ballia to Ibrahimbad has since been completed, and a protective bund along the right bank of the Yamuna in Panipat is in progress and armoured spurs are being constructed for protection.

7. Flood Relief: From the devastation by floods people suffer very badly. All their belongings, movable and immovable, are washed away by floods, and the people are left homeless and penniless. Immediate relief measures are therefore necessary. The usual things that are thought of as relief are food, clothing and shelter. These should be supplied to the people immediately. This is the concern of the Government (both State and the Central Governments.)

One important point that is not generally remembered is employment of the people, who are thrown out of their routine work by the floods. This is an important item, and Government

should immediately arrange to start relief measures and provide work for all the people in the affected area.

Conditions in each area affected by floods should be studied carefully and necessary steps should be taken to restore it to normal, and also to prevent the recurrence of the devastation by flood.

The works that have to be done in this connection are the protection of the towns specially and the protection of the whole area from the havocs of flood. The towns should be protected by properly designed embankments, as also the fields and the land on either bank of the river. The level of the towns may also be raised if possible to some extent wherever it is economical to do so.

The drainage of the country, in general, should be improved, so that pools may not be formed during the flood and cause epidemics in the form of Cholera, Malaria, etc.

Wherever the drainage cannot be improved as above, heavy expenditure has to be incurred by installing sufficient pumps to drain the water from the affected area.

Generally, floods are caused by the changing of the course of rivers, and adequate, careful and costly maintenance works will have to be done in the matter.

For Flood Control, the following information should be first collected,

(1) Intensive collection of data (both meteorological and seismographic) should be made, especially in the area of rivers.

(2) Collection of geological and botanical survey data should be made. In the Himalayan region this should extend up to the glacial region.

(3) Hydrological data of rivers should be got.

Collection of data of intensive soil survey, including mechanical and chemical analysis of the soils should be got.

(4) Close topographical survey of the country especially in the region of rivers should be made.

Hydraulic models should be prepared and the effect of the rivers should be studied on these models in the research institutes and necessary action taken to control the river.

Short term courses should be started to train overseers, who are required in good number, for the supervision of works.

A Commission should be set up for each river or a series of rivers, just like the Damodar Valley Corporation, or the Ganga and Brahmaputra River Commission; and to supervise or to control the work of these organisations, a central flood control board, which is already set up by the government, should be permanently maintained.

From the 26th Annual Meeting of the Central Board of Irrigation and Power on 8th Nov. 55, the following decisions were arrived at.

Flood Control : Advancement of Engineering Science has made it possible to find methods of reducing flood damage. They are

(1) Construction of Detention Reservoirs.

(2) Improvement to the streams or rivers to speed the flood water on their way.

(3) Devices to transfer the flood water to newly-cut channels.

In addition to the above works, Meteorological surveys of the Himalayan region about

(a) the variation of monsoon and

(b) Melting of snow have to be very carefully taken and studied.

(A) Flood Prevention :—This is effected by (1) Storage dams and reservoirs, (2) Detention Basins, (3) Ground Water Storage, and (4) Afforestation.

(1) *Storage Reservoirs and Dams* :—On a large river, when there are uncontrolled floods destroying towns and agricultural tracts, a flood regulating reservoir is necessary. The capacity of the reservoir should be so great, that it never attains the high flood level, and such that the floods pass through them with the same intensity, but its power for damage will be reduced in this duration. This type of reservoir can be constructed only to store the maximum flood yield, and must be provided with large sluices with high discharge, to run off the storage in the interval between the floods.

When the reservoir water is not utilised for any purposes, (the reservoir being on the main stream), the sluices need not be provided; instead, the vents can be left constantly open. Sluices are only required when the storage is to be utilised for irrigation, etc.

If storage dams are built of required capacity, they will have undoubtedly the effect of moderating the floods. For this purpose, they should be kept empty at the beginning of the rainy season, so that the flood may be absorbed in the reservoir. This is not by itself an economical problem, and so it is combined with other schemes, namely, Hydro-Electric, Water Supply, etc. The situation of these dams has to be carefully selected, not merely for engineering and geological features, but also with regard to the distance of the dam from the flood affected area.

(2) *Detention Basins* :—Openings are left in the dam and these are allowed to function constantly ; also they are automatic and do not depend on any body's discretion. They retard the flow by partial absorption and allow safe flood downstream. These are used on a large scale in France. These are what are called " Detention Basins. " This is a flood control reservoir with its outlets under control. This type is very useful when the area to be protected is large and widespread.

Its advantages are (1) A uniform discharge can be allowed through the outlets.

(2) Even though the rainfall and run-off vary in intensity over the whole watershed, the discharge can be regulated as and when required.

(3) If there is flood in one stream, the gates in the reservoir across the other stream may be kept closed, until the flood situation is cleared.

There are also retaining basins with auxiliary gate control. This type consists of both controlled and uncontrolled gates. This is to allow a more rapid emptying of reservoir, where the storage capacity is large (7 in. run-off) as compared with its drainage area, with a correspondingly small conduit capacity.

The size of uncontrolled outlets is so small, that the emptying period is unduly long and hence control gates are also provided. Capacity of flood control reservoirs varies between 4 and 9 in. of run-off of the drainage area.

The discharge through the outlets is designed to be less than the full capacity of the stream downstream, and these outlets are not controlled and they work automatically. The *advantages* are

(i) No gates are provided and human elements are eliminated.

(ii) Savings in the cost of the gate and its installation.

The *disadvantages* are

(i) There are no gates to control the floods.

(ii) There may be coincidence of flood crests further down-stream.

(3) *Ground-Water-Storage*: This is secured by forming terraces in the catchment area and by raising crops which absorb rainfall to a certain extent. The effect of this is the reduction of the run-off. This method is adopted in T. V. A. and it is said that the absorption is even 4 in. run-off.

(4) *Afforestation*:—This has a retaining effect on the run-off and it ensures better regularities of flow during the high flood season. It prevents erosion of steep mountain sides—an important factor on flood prevention.

5. (B) Flood Diversion.—This is chiefly diversion of the flood, from places where damage is likely to occur, to less objectionable places. It is effected by high level escape channels. For this purpose, the head of the diversion channel should be located on the convex side of the stream and the velocity throughout the channel must exceed the velocity in the stream, immediately opposite the diversion channel.

It is advisable to provide a regulator either for the river or the diversion channel at the junction. In deltaic regions, where the river takes a circuitous course, a direct route to the sea must be got excavated; but it must be seen at the same time, that the sea water does not flow into the river.

6. (C) Flood Protection:—Levees (embankments) are more extensively used than other means, mostly in America. In India also, this is in use in Madras, Orissa and Bengal. Levees are formed of the soil available at site, namely sand and silt. Being porous, the banks get saturated very soon, and consequently slips are very common. To get over this disadvantage, the embankments are made broad and the slopes sufficiently flat and also turfed over. Percolation drains are also provided to strengthen the base. The height of the embankment is generally limited from 20 to 30 ft. These should be designed properly, especially regarding the foundation, the nature of the soil available for the formation of the embankment, the duration and the stage of high flood; and

the margin regarding free-board in the city area and near agricultural lands should be specially studied for their proper design.

As they are subject to scouring action by the moving currents of water, they are protected by rough stone revetment or flexible concrete mats; they are adopted where adequate waterway cannot be secured.

Levees cannot be solely depended upon, and accordingly they are supplemented by diversions, by means of spillways. Such steps are adopted in the Mississippi river communications. The main disadvantage in levees is that they cause greater danger than the river by itself would have done.

In cities and urban areas, masonry walls are built in place of levees, to save space and to have less risk. These can be managed with less cost.

River Channel Improvements :—These are generally combined with levees and they are generally effected by

(a) Widening and deepening (increasing the sectional area of the river).

(b) Straightening the river channels.

(c) Smoothening the channel surface, by the removal of obstruction; (flood water is used to clear the dredged material).

(d) Guiding the currents, where river divides into branches, so that each branch takes its share of flowing water and its silt also (regulators may be constructed at the forks).

7. Model Experiments: Model experiments should be conducted at Research Stations and their effect tested; because, it is seen in some cases that the effect is only of a temporary nature. As the cost involved is small, the experiments are worthwhile doing.

8. Flood Warnings: Warnings should be given in advance by (1) Telephone, (2) Radio, and (3) Telegram. Radio is very useful during floods, as the other two are liable to fail during heavy storms.

9. General: Flood control works should be of a permanent nature. They should be spread over many years, so that the burden be shared by the existing and the future generations also.

As experience is gained, better remedies are also employed. The cost of protection should be of a certain proportion to the cost of damages averted. The Central Government should bear the major portion of the cost, and the people also should be made to bear some cost. Patient study and research work only can solve the problem of countering the floods of great rivers. The necessary main works to be done are

(1) An adequate system of run-off and rainfall run-off must be installed.

(2) A system of communication to get information to the centre must be introduced.

(3) A basic plan of the operations must be installed at each reservoir.

(4) The operating personnel should be selected.

Regarding flood control in the river Brahmaputra, it is said that unless embankments are built along all the streams, i. e. the main river and all its tributaries, it will be futile to try to tame only the main river through dykes. Moreover, the river is always changing its course at convenient places.

(II) FLOOD DISPOSAL

For any storage reservoir, proper provision should be made, to pass off safely, all the surplus flood water that cannot be impounded in the reservoir itself.

The devices employed for the flood-disposal are

1. Spillways or weirs.
2. Crest Control.
3. Crest Gates.
4. Spillways and Sluice Gates.
5. Spillways and Siphons.

If the reservoir contains only a spillway, the discharge over it is fixed by the length of the spillway and the depth of discharge allowed over its crest. This depth cannot be increased easily, as in many cases a large area of land may be submerged in the

water-spread area of the reservoir; or it may encroach upon the rights of private properties. Hence the length should be increased to get the required discharge for the surplus flood. This length cannot be economically got in all cases; hence in addition to the weirs or spillways, sluice gates or/and syphons are installed, to supplement the discharge over the weir proper. Examples of these can be found in the Krishnarajasagara (Mysore), Bhavanisagar (Madras), etc. For details of these and similar reservoirs, see.

1. **Spillways**: These are already described in previous chapters. Weirs of tanks in Chapter XII and River weirs in Chapter XIII.

2. **Crest Control**: These are also called "*moving weirs*". They are used for lowering or raising the crest level according to the river discharge. They may be of the following types:

(a) Flash boards.

(b) Drum gates.

(c) Tilting gates.

(d) Bear trap gates.

Only Flash boards are in general use and are used to store the extra water of the flood temporarily, and the water level is lowered either automatically or by manipulation.

3. **Crest Gates**: These are used to partly control the fluctuations of the flood flow, and where water is valuable the type which is watertight and does not allow any wastage of water is adopted. If there is any fear of ice or floating debris, the vent should be unobstructed, or if piers are built they should be wide apart. The gates are fixed between the piers constructed on the crest of the dam, generally at regular intervals. On the top of these piers, a bridge is laid and a hoist on the bridge operates the gates.

Examples of this device are (a) Taintor Gates and (b) Stop logs and needles, which are explained separately below.

4. **Sluice gates**: These are placed not on the crest of the dam but in the lower levels (of the dam). These are used as stated

above—in addition to the ordinary spillway. The weight of the gates when raised and the water-weight will both have to be carried on the piers, which support them, and hence they will have to be designed properly.

The gates are operated mostly by power, and sometimes when the number is small by hand, or they may be automatic.

Examples of this are (a) Sliding gates (b) Butterfly gates, etc. which are described separately below.

Sliding gates There is a lot of friction here, due to the water pressure on the surface at the ends of the gate. Hence these gates are limited only to small sizes.

Roller Bearing Gates: These are an improvement over the previous ones, as cylindrical rollers are introduced to minimise frictional resistance while raising or lowering the gate.

Stoney's gates are a fine example of this type, and is already described in detail in a previous chapter (Chapter XII, on Sluices and Weirs).

Syphon Spillways: These increase the capacity of the reservoir by ponding, and there is in addition to the head on the crest, the suction head. Detailed description of the different kinds of spillways is given further down in the chapter.

GATES

1. **Introduction**.—Crest Gates for waste weirs are auxiliary installations employed to exploit the storage capacity of the reservoir between the full reservoir level and the maximum flood level. It is often the case, that the waste weir left to itself during floods is noticed to discharge with a considerable head over it. The crest level of the weir being at the full reservoir level, this extra water corresponding to the storage between the full reservoir level and the maximum flood level is wasted, although the storing of it may not affect the stability of the reservoir, but raising the solid weir itself up to the maximum flood level is forbidden, as it puts a permanent obstruction to the flood and has the effect of raising the flood levels above its new crest level. Enhancing the storage nevertheless has a great significance where water is precious.

The gates provided may be either (i) Non-automatic gates or (ii) Automatic gates, in their working.

2. Non-automatic gates :—Non-automatic gates are gates operated by manual labour. The gates have to be opened by manual labour, when the level of water in the reservoir rises above the full reservoir level; this is a disadvantage during sudden floods. There are different types of non-automatic gates, and 3 types in common use, viz. Taintor Gates, Drum Gates and Butterfly Gates, are described below.

3. Taintor Gate .—This gate is frequently used to regulate the surplus water in storage reservoirs, where the distance between the piers on the crest of the waste weir is large. This gate is also called *Sector Gate*.

The gate consists of a circular sector bulging towards the water face supported by means of two struts leading to a pivot at the centre of the sector, the struts being placed one at each end of the span, parallel to the piers, on the crest of the waste weir (See Fig. 216). These gates are made usually of structural steel throughout and in some cases for small gates steel circular sectors covered with creosoted wood are used.

TAINTOR GATES

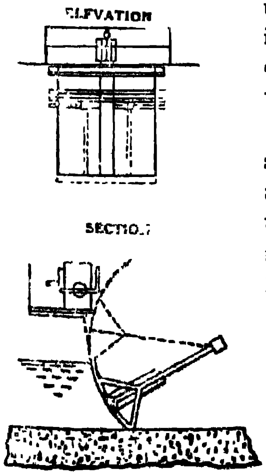


Fig. 216

The gates are operated by chains from a drum fixed to the bottom of the gates at each end of the span. Since the angular travel to the bearings is small when the gate is raised from the closed to the open position, the work done in overcoming friction is small. The operating force has the advantage of leverage over the frictional resistance at the bearing.

The force required to overcome the bearing friction is

$$P = \frac{KW_{\rho r}}{R}$$

where P is the force required

K is the coefficient of friction

W_p is total water pressure on the gate

r is the radius of bearing

and R is the radius at which the force P is applied.

Taintor gates are used where the floods are of moderate height. There are massive trunnions or bearings provided on reinforced cement concrete cross beams on which the gate rotates. Since the gates are hydraulically counter-balanced (the balanced water pressure) on their faces, they readily close. The gates will be above the water level, when they are open.

In some cases, the gates open downwards and are lowered into a recess below the crest and when raised, their tops will be at the same level as the spillway weir.

(4) Drum Gate.— This type of gate consists of a number of sectors of a cylinder of angle 45° to $67\frac{1}{2}°$ provided between the piers on the crest of the waste weir. (See Fig. 217) Every beam of this structure appears as a built-up plate girder and is slightly bulging outwards. There is a small angle between the vertical and the beam to the side of the water. The face of every beam is covered with steel plates in the form of a drum. The gate is hinged or pivoted at the centre or at the top-most vertex, to the crest of the waste weir, and on this the gate rotates.

DRUM GATE

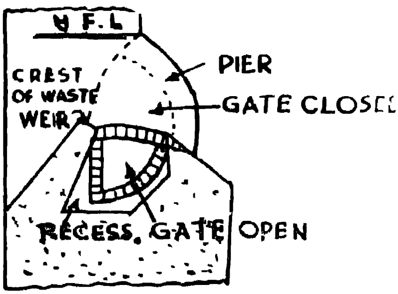


Fig 217.

On the downstream side, the gate contains a watertight chamber slightly bigger than the gate, so that it may accommodate the gate freely, in its open position. In the open position of the gate, the face to the water side acts as a cover to the chamber and allows the water to overflow over it.

By varying the head of water in the chamber, the gate is raised or lowered, and special valves are provided for the same. A counterpoise is generally provided in case of large spans for non-automatic gates, as it requires less energy to work it.

(5) Butterfly Gate or valve: Generally the gate of this type is provided inside a pipe. It consists of a circular disc exactly fitting into the pipe, and an axis passes through the centre of the disc (See Fig. 218). When closed, the gate leaf can be riveted into position by vertical gate seats which are adjustable with tapered surface. The gate leaf rests on a thrust bearing mounted on the lower pivot. The gate is operated by a crank acting through a worm and worm wheel.

BUTTERFLY GATE

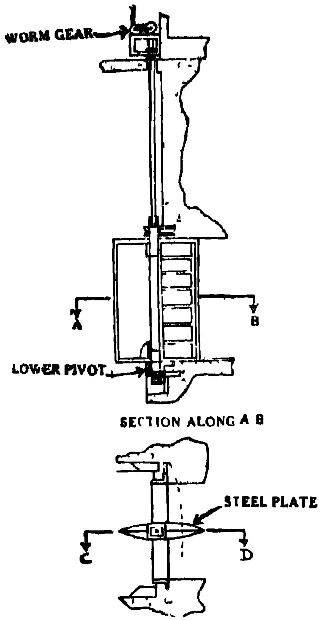


Fig. 218

The gate is balanced under hydrostatic pressure, either when the gate is open or when it is closed, since the axis passes through the centre of the disc. For every alternate position, the gate will not be counterbalanced by the hydrostatic pressure correctly, and as such this type of gate cannot be used for regulating the flow of water under high heads, but it is frequently used for full closed or full opened operations under high heads also. For low heads this type of gates is most suitable. These are used both as guard gates and as opening valves under moderate heads.

Advantages are that they require only relatively small space outside of the line of sluice (when used as a guard gate).

6. Automatic Gates:—Automatic gates are devices by which weirs are fitted with gates which open automatically when the flood levels rise above the maximum flood level, and leave the weir to discharge freely with its usual head, and automatically close, when the water level in the reservoir falls below the full reservoir level. It, in fact, functions as a device to maintain the level of the reservoir at the maximum flood level during floods irrespective of the discharge.

All automatic gates contain invariably the same floating device. In principle, the automatic gates are nothing but weir gates counterbalanced by a dead weight, sufficient to lift the gates. The counterweight works in a water-tight chamber. When there is no water in the counterweight chamber, the counterweight sinks under its own weight, and pulls the gate upwards, whose movements it controls and closes the sluice ventways. When the counterweight is lifted up, the gate moves down, thus opening the ventway.

An inlet pipe provided for the counterweight chamber, has its bottom above the full reservoir level, and it draws water into the counterweight chamber when the water level in the reservoir exceeds the maximum flood level, and gets filled with water. When this happens, the counterweight suffers loss of weight and rises up, as it can no longer support the gate, which consequently falls down causing the discharge.

The automatic closing of the gate is effected by an outlet pipe, which is exhausting all the time. As long as there is inflow into the counterweight chamber, there is residual water filling it, but when once the flood level comes down, there is no inflow. The outflow continues and the chamber gets fully drained in a short time, and the flotation ceases, the reverse operation taking place.

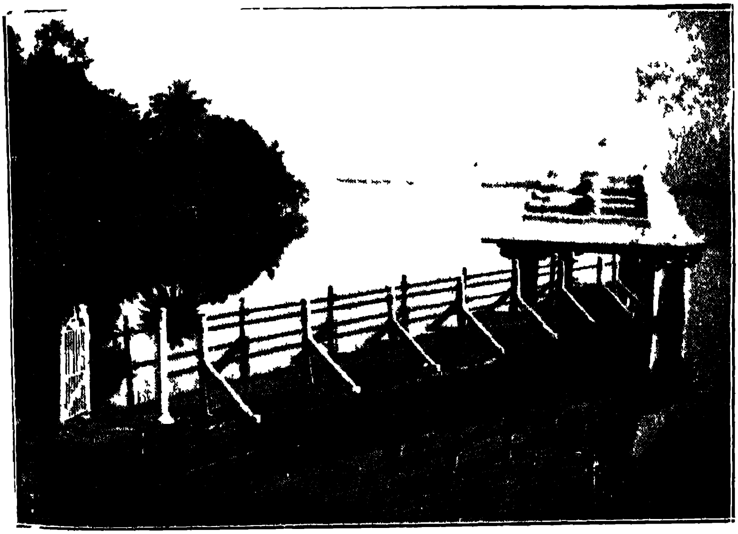
7. Advantages and Disadvantages of Automatic gates:— The *advantages* are,

(1) The capacity of the reservoir or the storage capacity is increased, though temporarily. (2) The gates work automatically and hence there is no dependence on manual labour.

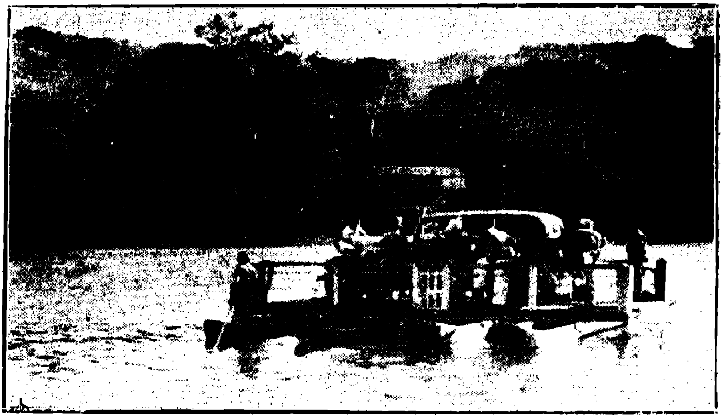
The *disadvantages* are :—

(1) Trained men are required to keep the machinery in good working order. (2) The gates are liable to damage by logs, etc., during floods. (3) During working operations, the gate may stick up, and to lift the same, accessories like crane, etc., will have to be got. (4) The gates cost more both initially as well as for maintenance. (5) Even with the best type of gates, there is always a good amount of leakage. (6) When the automatic gates work, they discharge suddenly a large volume of water which may endanger men and property lower down the valley.

N. B.—As a precautionary measure, it is always a good policy to have a stand-by, in place of an automatic gate. That is,



Foot Bridge of an irrigation sluice of a Reservoir.



Car carried on boats

Car Landing at the Ford

provision of enough open weir, (which is safer than the gate), should be made.

There are different types of automatic gates in India, which are patented by eminent Engineers.

8. Reinold's Gate :—This is an automatic type of roller gate. Usually the gates are 10 ft. long and 8 ft. wide. The gates move on rollers running on rails. The sluice frame is fixed at a very small angle to the gate, in order to have a free motion and watertight connection between the two, when the gate is fully raised. The counterweight is suspended by two chains passing over a system of pulleys on each side. (See Fig. 219). The weight of the counterweight is greater than that of the gate when both are weighed in air, but its weight in water is less than the weight of the gate in air.

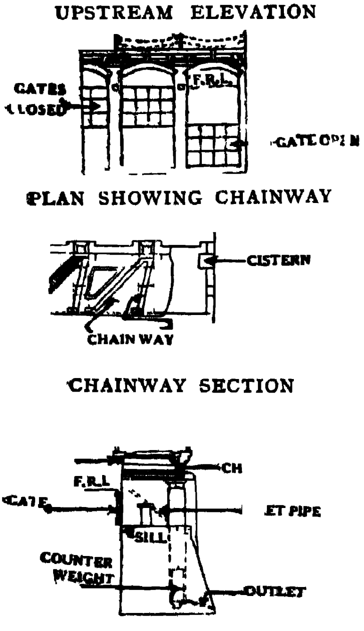


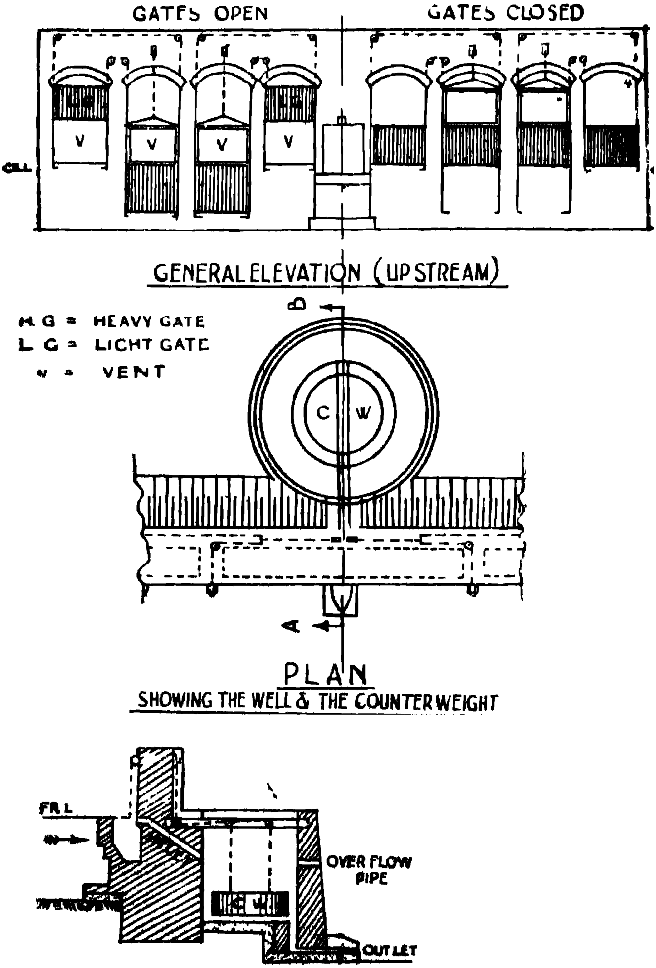
Fig. 219.

The mouth of the inlet pipe is about 12" × 12" and is covered with a net cover to prevent any muddy water from entering the counterweight chamber. The gate works as described above.

9. Visvesvaraya's Gate:—This is a fixed-roller type of automatic gate and is similar to Remold's gate. The gates work in pairs and are hung by chains. Each pair contains a lighter gate and a heavier gate, and as such, the counterweight which has to be attached to them in common, has to work against only the difference of weight between the two gates. The weight of the counterweight is such, that it is sufficient to pull the heavier gate upwards to the closed position, which in turn, makes the lighter gate to fall down by its own weight and close its sluice vent also. The heavier gate falls down pulling

the lighter gate upwards, thus opening both the sluice vents when the counterweight rises, losing weight in its chamber, when it gets filled with water.

UPSTREAM ELEVATION OF VISVESVARAYA'S AUTOMATIC GATE



SECTION AB.

Fig. 220.

This type of gate is installed on the waste weir of Lake Fife, Poona, and its description is given below:—

There are eleven sets of gates, each set containing eight gates, of which four gates are heavy and the other four are lighter. (See Fig. 220). The counterweight is connected to the four heavy gates in each set and it works in a watertight chamber called counterweight well. This well is provided at the back of the waste weir with four gates on each side, containing two pairs of heavy and light gates. These gates work between the piers (and the thicker pier is at the centre of the eight gates.) The heavier gates are connected to the lighter gates by chains passing over a system of pulleys.

The heavy gates are 10 ft. long and 8 ft. wide and are provided in the frame work at the top of it. At the centre of the top member of this framework, starts the chain connecting the counterweight and the heavy gate, and this arrangement prevents the chain from being in the way of discharging water, when the heavy gates are open. The inlet pipe has a mouth of about $12'' \times 12''$. The working of the gates is as explained earlier.

Another gate patented by Mr. Visvesvaraya consists of gates of equal weight, and the waste weir is put into work, when they are raised. An upper balance weight works in a chamber constructed in a sluice pier, and is connected to each gate by chains. The weight of it is not able to open the sluice vent, as it is not sufficient to overcome the resistance of the gate and the counterweight in dry, which is connected to the gate by chains and works in a watertight chamber as before. The gate falls and closes the sluice vent, due to the weight of it and the counterweight in the dry. The balance weight pulls up the gate and opens the sluice vent when the water enters the counterweight chamber and the counterweight rises, losing weight.

10. Shaft spillways. A flaring funnel attached at the top of a down-take pipe called the shaft mainly constitutes the Shaft spillway. The top of the funnel is at the full reservoir level and is called the crest of the spillway. A right-angled bend is given at the bottom of the spill-way. To discharge the surplus water on the downstream side, a horizontal tunnel is provided beyond the bend, under or around the side of the dam. Water flows over the lip of

the funnel and discharges down through the tunnel. The discharge of a shaft spillway is given by the formula $Q = CLH^{\frac{3}{2}}$ c. ft. per sec.

where Q is the discharge of one swallow hole, in c. ft. per sec.

C is a constant for the shaft spillway.

L is the periphery at the top of the funnel in ft.

H is the depth in feet of flood water above the crest.

The number of units of swallow holes to be provided for the spillway is determined by the ratio of the high flood discharge to the discharge of the shaft spillway. Two or more swallow hole units are provided when the high flood discharge is large.

Shaft spillways are classified according to the shape of their crests. They are:

(a) Standard round-crested Shaft spillway (See Fig. 221).

(b) Flat-crested Shaft spillway (See Fig. 222).

STANDARD ROUND CRESTED SHAFT SPILLWAY.

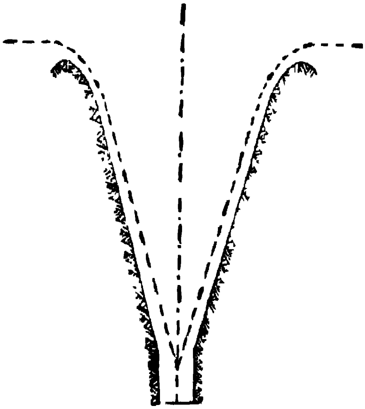


Fig. 221.

FLAT CRESTED SHAFT SPILLWAY.

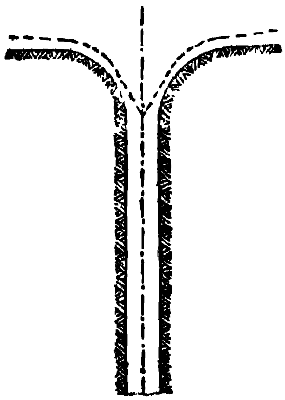


Fig. 222.

The *disadvantages* of the Shaft spillway are:

(a) The discharge increases slightly, with the depth of overflow, beyond a certain limit.

(b) Against under-estimation of floods, the shaft spillways do not give a great factor of safety.

(c) There will be always trouble at the bend.

(d) Under high heads, the shaft spillways are dangerous.

Since the length of the crest should be enough to accommodate the discharge at the maximum head permitted, the shape of the spillway depends upon the discharge and the depth of overflow. The discharge and the fall determine the size of the outlet tunnel. The tunnel is generally constructed to flow throughout its full length, and when there is a maximum discharge, it should not produce a backwater action on the crest of the spillway.

Shaft spillways are generally used in Arched dams. They are located on the upstream side, towards a side of the dam.

Shaft spillways are best suited to canyons where the room for the spillway is restricted.

11. Syphons:—*Introduction*:—The flood discharge over an ordinary weir is of some appreciable height that a good area of land is often submerged by it. Also, protective works are required when the flood falls from the top of the dam on the outer toe. Again, the cost of excavating a draft channel is great. Hence a device called "*Syphon spillway*" is sometimes adopted in place of the ordinary overfall weir.

The syphon spillway utilises the available head to produce a higher coefficient for the flow of discharge than would be obtained in the case of the usual overflow type of weir.

Classification of Syphons:—Syphons are classed as (1) Hood syphons, (2) Volute syphons.

12. Hood Syphons:—For the syphon to work satisfactorily a vacuum should be created for the discharge through the full bore of the syphon tube. There are various methods to create this vacuum, namely, (i) exhaust by air pump, (ii) adoption of a stepped weir, (iii) baby syphon, (iv) method of deflecting plate, and (v) priming weir. The method that is generally used is the priming weir. In this type, the body wall in the syphon is so shaped that the overflowing *nappe* shoots across the syphon and seals the lower limb.

Requisites of a good syphon are:—(1) Low priming depth. (2) Fast priming. (3) Higher coefficient of discharge. (4) Smooth flow of water.

The priming depth is defined as the head above F. R. L. which causes sufficient vacuum to be created to make the discharge full bore at the outlet.

SYPHON SPILLWAY

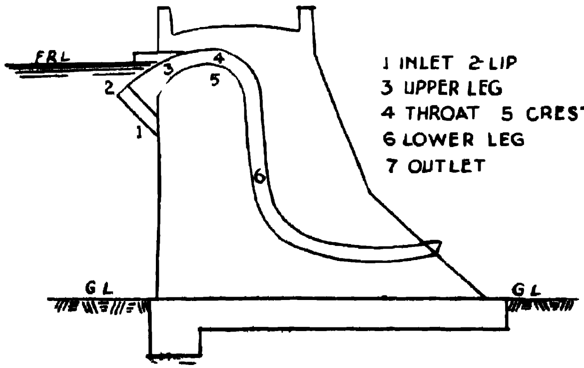


Fig. 223.

The lip of the funnel of the syphon which is kept at F. R. L. is so adjusted that it seals the air outlet when water rises above F. R. L. and thus syphonic action commences. (See Fig. 223).

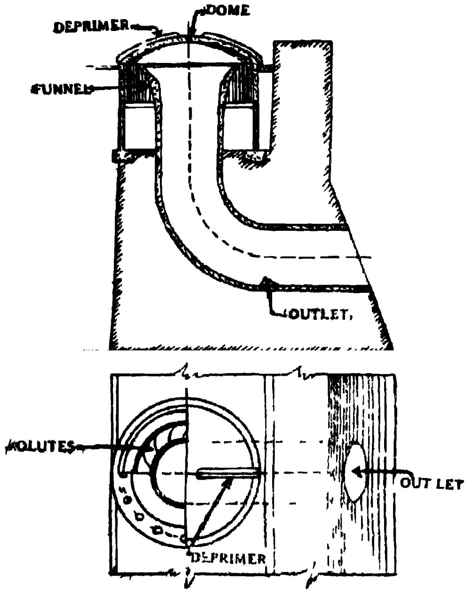
As soon as water rises above F. R. L., the air inlet is submerged and so outside air is cut off from it. When the water spills over the crest of the syphon, this water takes away with it some air, and as more water rushes in, vacuum is created, and the water rushes full bore into the tube of the syphon. When the water level *falls below F. R. L. the outside air rushes through the inlet into the syphon, and its action is broken.*

(13) Volute Syphon:—This consists of a dome with a funnel placed underneath it with an annular space in it. The funnel is kept on the top of the dam with its lip at F. R. L. A number of volutes are placed radially, and they induce a vortex motion to the discharge of water. A vertical pipe is taken from the funnel through the body of the dam and is connected to a horizontal pipe leading outside the dam. This vertical pipe is covered

with a dome resting, as said above, on pillars on a platform around the funnel. The dome prevents any floating debris entering into the syphon. (See Fig. 224).

When the water level rises above F. R. L. it spills over the lip of the funnel and descends into it, following in radial filaments parallel to the direction of the volute. These meet in a vortex and on account of the centrifugal action, the vortex sucks in the air, and a vacuum is created and syphonic action starts immediately.

SECTION OF VOLUTE SYPHON



Half plan at the top and half plan dome removed
Fig. 224.

When the level falls below F. R. L. air enters through the air pipe into the dome, breaking the vacuum and the syphonic action.*

The one drawback said of the volute syphon is, while water enters through the volute into the body of the syphon, it takes the path as shown in Fig. above leaving some empty spac

* Air Inlet pipes are placed on the top of the dome, one end ...ar with F. R. L. and the other end inside the dome at centre.

between it and the side wall of the syphon. These cavities are said to be filled with dissolved air and water vapour, which after sometime will burst, causing a deleterious effect upon the structure on the sides of the syphon. This effect is still under watch and final investigations will have to be made.

14. Advantages:—(1) They are entirely automatic and fool-proof in action.

(2) There are no working parts to go wrong, or out of action.

(3) They reduce flood lift and thus add to the capacity of the reservoir, and that too, in upper contours which are very large. Thus they reduce submersion and admit the top of the bund or dam being kept lower by the amount of the reduced flood lift. Hence, a large saving is effected in masonry and/or earthwork.

(4) They are brought into full action at once.

(5) The length of the weir required is considerably less than in the case of an overflow type.

(6) They involve no operating cost.

(7) They permit close regulation of water level.

(8) The priming depth required is small. It is seen that to get a discharge of 10,000 cusecs, only 18 in. priming depth is necessary, in place of 2½ ft. required for a hood syphon.

(9) They do not discharge debris or ice over the dam as the inlet level is below the top of the dam.

(10) They can also be used for scouring silt from the bed of the dam, (especially during floods).

(11) Only local labour and material can be used for the work, and so it is economical.

15. Discharge; The discharge through the siphon is given by the usual formula;

$$Q = CA\sqrt{2gh}$$

where C is a constant,

A is the area of syphon,

h is the operating head and

C is taken as

the 0·65 for hood syphons, and

zont 0·80 for volute syphons.

ARRANGEMENTS FOR FLOOD DISPOSAL

(A) Krishnaraja Sagara Dam: (a) 40 Vents of 8′ × 12′ with their sill at R. L. 106·00 provided with lift gates and worked electrically by a travelling crane.

(b) 48 Vents of 10′ × 8′ with their sills at R. L. 103·00. Also worked as above.

(c) 48 Vents of 10′ × 10′ with sill at R. L. 114·00. These are provided with automatic gates, which are placed in 6 batteries of 8 gates each.

These gates open automatically above R. L. 124·00 and close below that level.

(d) 16 Sluices of 10′ × 20′ with sill at R. L. 80·00. Each is provided with a gate operated electrically, by an independent crab-winch.

(e) 8 Sluices–Scouring–of 6′ × 12′ at sill at R. L. 12·00. These are for passing heavy floods. The gates are operated mechanically, by independent crab-winches.

(f) 3 Sluices of 6′ × 15′ with sill at R. L. 50·00.

Situated in the North Bank between the turbines and the Visveswaraya Canal. These are worked mechanically by crab-winches. Maximum discharge through them is 3,50,000 cusecs.

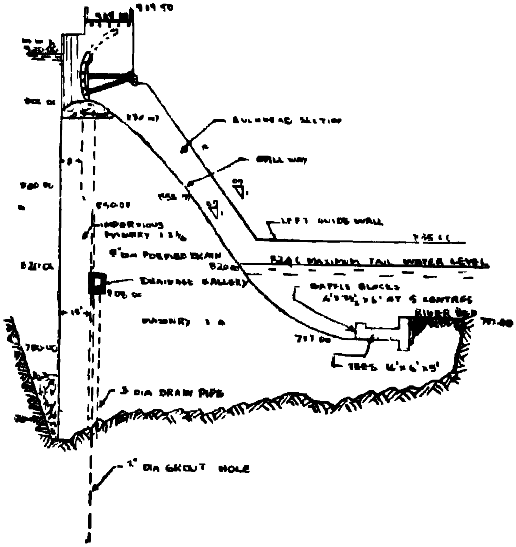
(g) One small Sluice of 4′ × 8′ in the North Bank, originally intended to feed the Low Level Canal, can also be used for floods.

(B) Bhavani Sagar: In the river portion, an Ogee type of spillway of length 396 feet is provided to discharge a flood of 1,22,000 cusecs. In addition, the dam has nine river sluices of 6′ × 10′ for passing some portion of the discharge. Thus it is seen that the spillway, together with the river sluices will permit a discharge of 1,56,700 cusecs.

Spillway: A section of the spillway as constructed, is given in Fig. 225. The length of the spillway is 396 ft., as already mentioned. The top of the spillway will be provided with nine Radial Gates of 36′ × 20′, with the full raservoir levels at the top of the shutters.

The floods will spill down a height of 113 feet. In order to dissipate the energy of the falling sheet of water on the rear side of the spillway, a stilling basin of required dimensions with

baffle blocks of size 7′ – 6″ × 4′ – 0″ × 6′ – 0″ at centres 22½ feet apart is provided. This arrangement for dissipating the kinetic energy of the falling sheet of water was evolved in the Poondi Research Station, Madras, and this was found to be satisfactory.



Spillway–Bhavanisagara.
Fig. 225.

(C) Spillway at Panchet Hill Dam (Damodar Valley): The spillway consists of 15 crest gates and 10 under-sluices. The upstream face of the spillway is vertical and the downstream face follows a curve of the shape of the nappe, ending in a bucket or stilling pool.

The crest gates are of radial type, having a travel of 63°–30′ with the trunnion axis. Drainage and operation galleries are also provided.

(D) Spillway at Assuan Dam (Egypt): This is the only solid dam, which passes the full discharge of a huge river like the Nile, solely through its body. For this purpose,

10 Low Level Sluices of 23′ × 6½′
and 40 High Level Sluices of 110′ × 6′

are provided; and these discharge 5 lakhs cusecs, and the velocity in the Low Level Sluice is 20 ft. per sec.

The rise and fall in the river Nile is very regular, and hence the sluices are not generally strained.

16. Fish Ladder: A fish ladder consists of a reinforced concrete passage for water, between the upstream and the downstream levels of water in the river, inclined at 3 : 1 or 4 : 1 divided into pools separated by baffles or divide walls with holes in their structure for the passage of fish. (See Fig. 226). The pools are usually about 4 ft. × 6 ft. and are so arranged that the water level in the successive pools from the bottom does not exceed 1 ft. to 2 ft. higher than the preceding one.

Design:—The slope of the trough should be 4 to 1 or 5 to 1. The velocity allowed is 10 ft. per second minimum. The change in the direction (staggered motion arranged) is for the fish to take rest.

The design should provide for as much friction as possible, so that the slope of the ladder will be a minimum for a given velocity.

The size and type depend upon the type of fish.

The objects of providing a fish ladder are

(1) To have a free movement for the fish along the river uninterrupted by

(a) Construction of weirs

(b) The velocity of water in the river. The maximum velocity against which fish can move is about 10 to 12 ft. per second, therefore the fish ladder should be so designed, that the velocity does not exceed this limit.

(2) To allow the fish to move from the upstream side to the downstream side of the river and vice versa.

The most suitable site for the location of a fish ladder is near the divide wall, since the downstream of the under sluice in the river bed has water throughout the year. Usually the fish ladder is provided between the weir and the divide wall; in some instances, it is provided within the divide wall itself.

Oblique walls staggered with holes are built so that the fish can take rest after passing from one hole to another. Shutters are provided at the uppermost and lowermost cross walls of the fish ladder, to control the flow of water through them.

17. Classification: There are three types of fish ladders in general use.

(1) *Sluice type:* Here the baffle board contains an opening (as shown in Fig. 226) for water to pass. These openings are staggered.

SLUICE TYPE

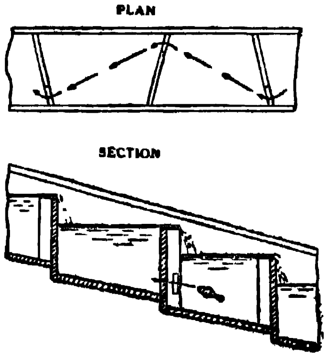
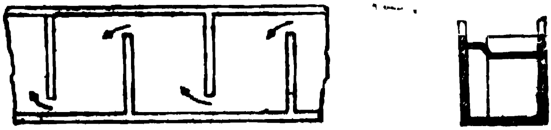


Fig. 226.

(2) *Rapid type*: In this type, the baffle walls are not to the full width of the trough and water passes through the opening left at one end. (Vide Fig. 227). These openings are also staggered to lessen the velocity.

RAPID TYPE

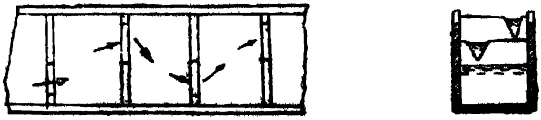


Plan Fish Ladders Section

Fig. 227.

(3) *Overflow type:* Here the baffle board contains a V shaped notch over which the water discharges and the fall is low enough for the fish to jump safely. (Vide Fig. 228).

OVERFLOW TYPE



Plan Fig. 228. Section

N. B. This type is not suitable for all kinds of fish ladders.

Logway or Logchute: Where logging operations are going on a large scale, log-chutes are provided on one side of the dam, just near the fish-ladder. For the transportation of the logs, they are allowed to float in the river and are collected on the upstream side of the dam. To pass them to the downstream side, a passage is quite essential and should be provided. This is called *logway* or *logchute.*

Design. The bottom of the chute is made 2 to 4 ft. lower than the crest level of the spillway of the dam and the width convenient to accommodate the logs. The logway is provided with gates to discharge water from the dam, through the incline, to the lower side where a pool is created to prevent the impinge of logs with high velocity on the foundation below. The chute should be narrow—as narrow as possible—to get the maximum depth. The width depends upon the size of the log, and the rate at which the log should pass the dam. Vide Fig. 229.

LOGWAY

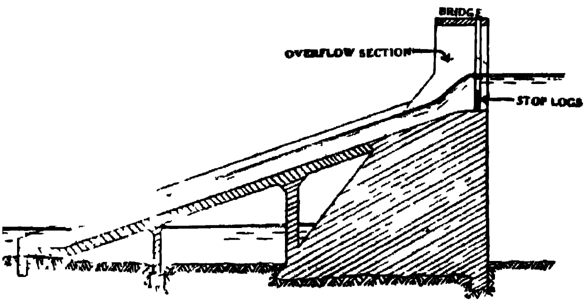


Fig. 229.

In some designs, provision is made to pass only one log at a time, while in others, more, as per requirements.

To guide the log into the chute timber booms are necessary and are provided for.

N. B.—In this type there is a lot of wastage of water, especially when water is required for power. Stop logs are very economical, but they can be used only for low heights of flood.

Gates are usually provided to shut the flow of water through the logway, when there are no logs. A pool is provided at the foot

of the logway on the downstream side to prevent any damage by the logs coming down the inclined logway.

18. Flash boards and Stop planks:—For controlling the water level in the reservoirs i. e. to raise the F. R. L. temporarily. Flash board or Stop planks are provided for the purpose. Flash boards consist of horizontal boards or planks of wood of convenient lengths and thicknesses supported on pins set up in sockets on the crest of a dam. (See Fig. 230).

These are safer and simpler in construction. They are cheap

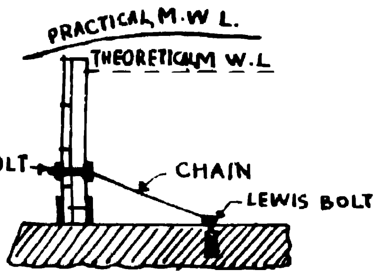


Fig. 230.

and can be replaced easily, though they and their supports are liable to be damaged by boulders. If they are left too long, and a flood occurs, the pins will be overpowered, when the water reaches a certain height and the boards will be released and washed away. The boards can be replaced only when the water level falls below their base. Many a time, the flash boards are lost during floods; hence they are arranged to be removed in advance.

Plain steel is the best material for the pins.—When the tension failure starts, the pipe collapses to the crest level and bends flat.

Permanent flash boards are similar in principle to the temporary ones; the chief difference being that the former is made to operate either automatically or manipulated otherwise, without any danger. Automatically-tipping supported flash boards which get released quickly in the case of sudden floods, are used in some cases.

In the case of stop planks, the boards are placed in slots provided in concrete piers in the reservoir, spaced at short intervals apart. (See Fig. 231). Since the force to be provided against is

STOP PLANK INSTALLATION

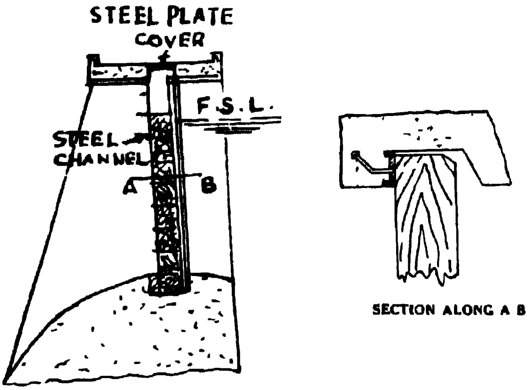


Fig. 231.

only water when it is in open position, and the forces are the water pressure on the flash board and the dashing of floating ice and debris, when the flash boards are removed, the designing of the piers and the steel bents should be in reinforced concrete and steel.

Needles. These consist of rows of timbers slightly inclined and supported at their top by a bridge or beam and at the bottom by a sill on the crest as shown in Fig. 232. The needles are placed one by one, by extending them horizontally over the pond and allowing them to tip into the current until the lower end swings down in contact with the concrete near the sill. They are rolled sidewise into places against those needles already installed, after they are drawn upward, until the lower end rests on the sill. The needles are hauled out bodily, lifting each from its seat in order to remove them. An anchor rope passes through a hole provided at the top of each needle, and this helps to hold the needle, in case it gets away from the operators when being handled. The needles used should not be too large as they will have to be installed and removed by hand. This type of flood control is not in general use. (See Fig. 232).

NEEDLE

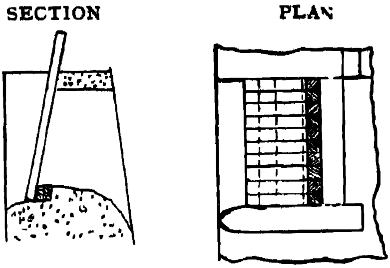


Fig. 232.

Questions

(1) Draw neat sketches showing plan, elevation, and section of Reynold's Gate.

(Dept. of Tech. Ed. 1954)

(2) Write a short note on Syphon Spillways.

(A. M. I. E. May 1951)

(3) Write a short note on Fish ladder.

(A. M I. E. November 52 & 53)

(4) Write short notes on (1) Shaft spillways.
(2) Movable weir crests.

(Bombay Univ. B. E. Apr. 1950)

(5) Write a short note on the Volute type of syphon.

(Poona Univ. B. E. Oct. 1952)

(6) (a) What are the advantages of waste weir gates used in storage dams ? What are their disadvantages ?

(b) Explain with dimensioned sketches any one type of automatic waste weir gates with which you may be familiar.

(Poona University, B. E. October 1952)

(7) Explain with sketches the working of the following and their use : (i) Fish ladder, (ii) Reynold's gate.

(Bom. Univ. B. E. Apr. 1949)

(8) Explain fully what is meant by Movable weir crests.

(Poona Univ. B. E. April 1954)

(9) Write a short note on :Volute syphon.

(Poona Univ. April 1954)

(10) What are automatic gates? What are their advantages? Explain with sketches the working of Visvesvaraya's gates.

(Bombay Univ. B. E. Civil. October 1949)

(11) Write a short note on Shaft spillway.

(Bom. Univ. B. E. Civil Oct. 1949)

(12) State the advantages and disadvantages of an automatic waste weir gate. Explain with the help of sketches the working of Visvesvaraya Automatic gates.

(Poona Univ., B. E. Nov. 1953.)

(13) Write a short note on fish ladder.

(Poona Univ., B. E. Nov. 53)

(14) Write a short note on Drum Gate.

(Bombay Univ. 53)

(15) Write a short note on Gates Spillway.

(Gujerat Univ., B. E. Apr. 53)

(16) Explain with sketches the function of:

(i) Fish ladder, (ii) Spillway. **(Gujerat Univ. 54)**

(17) Write a short note on Syphon spillways.

(Gujerat Univ. S. E. 53)

(18) Draw a plan, elevation and section of Volute Syphon.

(Gujerat Univ. B. E. Nov. 53)

(19) Distinguish clearly between Shaft spillway and Volute syphon spillway. **(Gujerat Univ. B. E Nov. 53)**

(20) What are the advantages of the syphon over other methods of disposal of flood waters? Describe with neat sketches the component parts of a volute syphon which you have seen during your tour and explain its working.

(Mysore Univ. Final B. E. Mar. 50)

(21) Write a short note on Syphon Spillways.

(Mysore Univ. Final B. E. Mar. 50)

(22) Write a short note on Volute syphon.

(Mysore Univ. Final B. E. Apr. 49)

(23) Under what situation do you adopt a syphon? Draw a typical plan and section of a syphon.

(Mysore Univ. Final B. E. 48)

(24) Discuss the conditions under which Syphon spillways are adopted. **(Mysore University Final B. E. 46)**

(25) Give a brief description, with the principles involved in the design, of a syphon spillway or a syphon in the body of a masonry dam. **(Mysore University Final B. E. 45)**

(26) Write a short note on Syphon spillways.

(27) Explain clearly the difference between lift and automatic gates; Saddle and Volute syphons.

(28) What are the different types of flood disposal works and what are the considerations which finally decide the particular type to be provided? **(Mysore Sept. 55)**

(29) Draw neat sketches and explain clearly their working.

(1) Volute siphon.
(2) Flood siphon.
(3) Visvesvaraya automatic gates.
(4) Fish ladder.

(Gujerat 55)

(30) What are crest gates? Describe briefly the three kinds thereof with special reference to

(1) Prevention of leakage of water.
(2) Frictionless movement.
(3) Emergency operation.

(31) Discuss the common methods of disposing of floods entering into reservoirs and discuss their advantages—what type do you recommend for an arch dam?

(Poona April 55)

(32) What do you mean by **Flood Disposal** in an Irrigation work? Explain.

(Mysore University, Sept. 55)

(33) Write short notes on syphon spillways.

Explain clearly the difference between saddle and volute syphons. **(Mysore University B. E. Civil Sept. 55)**

(34) Sketch any three types of Reservoir surplussing arrangements and explain how they function. Which of these types brings into play the flood absorption capacity of the reservoir?

(Mysore University, B. E. Civil Sept. 54)

(35) A reservoir is formed by a gravity dam 300′ high and 200′ long and the maximum discharge anticipated is 300,000 cusecs. Give a dimensioned sketch for the spillway arrangements. The F. S. L. and M. W. L. of the reservoir are to be the same.

(Poona University B. E. Civil April 57)

(36) Write explanatory notes on: Vishweswarayya's Automatic Gates. **(Poona University, B. E. Civil April 57)**

CHAPTER XV

CHANNELS

1. Channels:—The distribution works from a storage reservoir, through which the irrigation water is taken from the headworks to the fields, are called channels or canals.

These are clasified as follows:—(a) Main Canal or channel (b) Branch channel (c) Distributary channel (d) Field channel or *Hikkal* (See Fig. 233).

DISTRIBUTARY SYSTEM OF A CANAL

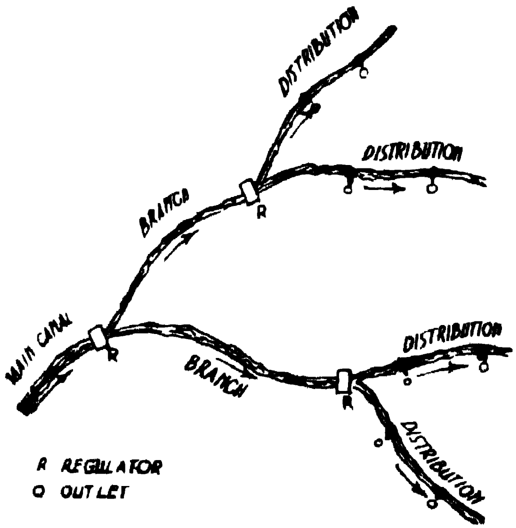


Fig. 233.

(a) *Main Channel or canal*:—This takes its supply directly from the headworks. There is no direct irrigation under the main canal generally.

(b) *Branch channel*:—A branch channel is an off-shoot from a canal, and even here, there is no direct irrigation under it. A branch channel is named after the important place through which it passes. For example, the Maddur Branch and the Cauvery Branch, taken from the Visvesvaraya Canal, Mysore.

(c) *Distributary Channel*:—This is a channel taking off from a branch channel or sometimes even from a canal, and from this, water is supplied to the field channels. Distributaries are given numbers to distinguish one from the other. For example, Distributary No. 1 and Distributary No. 2. etc. Sometimes a Distributary channel is also classed as (a) Major Distributary and (b) Minor Distributary.

(b) *Field Channel*:—This is a small channel excavated by the cultivator and also maintained by him. This is not the property of the Government.

N. B.—Generally the discharge in a branch channel is not less than 300 cusecs. The discharge in a major distributary ranges from 25 to 300 cusecs, while in a minor distributary it is less than 25 cusecs, and in a field channel the discharge is from 1 to 4 cusecs.

2. Selection of the alignment of a channel:—The following points should be considered before the final selection of the alignment of any channel :—(a) Cost of earthowrk along the route. (b) Cost of acquisition of the land required. (c) Cost of major distributaries. (If the channel is taken in contour this is saved). (d) If the route is made shorter (avoiding contour, at valleys and ridges), there is gain in head. Also, (i) some more area is got under command and thus the cost of the work is reduced, (ii) depth of cutting at any saddle further down is reduced and hence the cost of excavation is reduced.

But one should be careful while crossing valleys in deep embankments.

3. Layout of a Distribution System :—The chief considerations in the layout of a distribution system are : - (1) To secure effective water distribution most economically. (2) To get adequate command of the area to be irrigated. (3) To have practically no interference with the natural drainage.

Classification :—Irrigation channels are generally aligned with reference to contour of the country and are classed as :—

1. **Contour channels**
2. **Water shed or Ridge channels**
3. **Side-slope channels.**

(1) *Contour Channels* :—Here the channel is aligned along a falling contour, to produce the required velocity and the line of flow is at right angles to the falling ground. This channel cuts across the natural drainage lines. Hence the number of cross drainage works met with here are numerous.

To minimise the cost and lessen the length above, the channel is not allowed blindly to follow the contour, but is taken in deeper cutting along spurs and in embankments (even higher embankments if necessary) across valleys.

A canal from a reservoir should first be taken as a contour channel, and then, after some distance, taken as a ridge canal generally.

Final Location of a Contour Channel:—The area commanded by a contour channel varies with its longitudinal slope. The

CONTOUR CHANNEL

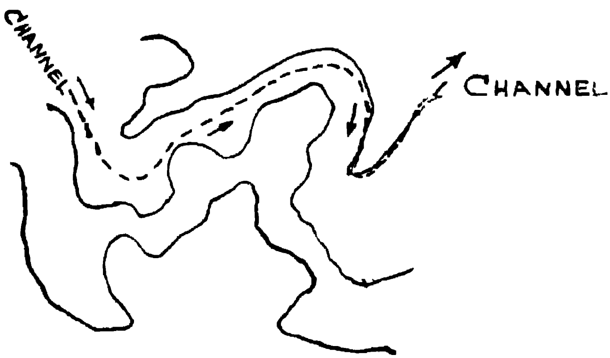


Fig. 234.

flatter the slope, the larger will be the area. The slope required to give a suitable velocity is dependent upon the capacity of the canal. This capacity can only be known from a rough estimate of the area commanded and the duty. From this data, a trial alignment of the canal should be got, and the area of the crose section of the channel roughly obtained and after some trials, ths final location adjusted. (See Fig. 234).

(2) *Water shed or ridge canal* :—This canal is generally taken along a ridge. The *advantages* here are (a) the channel does

not cross any drainage line, and (b) lands on both sides are brought under irrigation. (See Fig. 235).

RIDGE CHANNEL

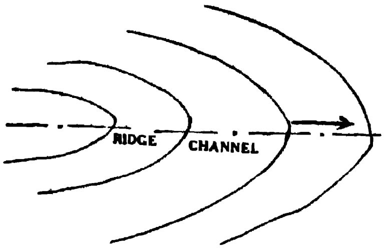


Fig 235.

(3) *Side slope canal* :—This is aligned at right angles to the contour of the country traversed. It runs parallel to the natural drainage and it avoids cross drainage works (See Fig. 236).

4. Banks—Single Bank and Double Bank Channels :— Channels are taken generally in, single banks, i. e., on the valley (lower) side. They are also taken in double embankments near the approaches of nallas where aqueducts are constructed.

SKETCH SHOWING CONTOUR AND SIDE SLOPE CHANNELS

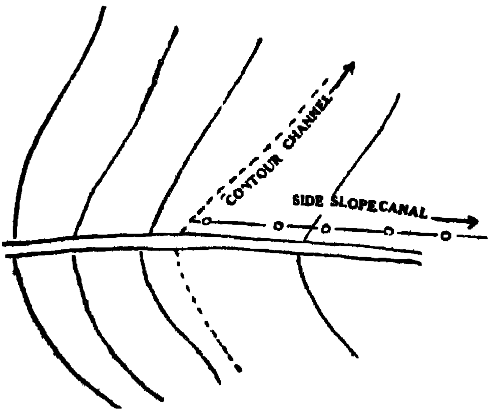


Fig. 236.

A single bank Channel is cheap in its initial cost, but it is liable to breaches, and also accumulation of silt, which is washed from the adjacent higher ground.

The double bank Channel prevents this silt entry into the channel, but its safety is always essential and proper precautions should be taken for the same.

Single Bank Channel :—These channels will have to accommodate the water draining into the canal from the side-long ground above. The regime of such channels is generally interfered by the silt brought into the canal. The maintenance charges here are high.

Examples :—Caddapah–Kurnool Canal, Upper Cauvery canal, and Tungabhadra canal.

5. Field Channels :—A number of fields are grouped together and water is supplied to them through one outlet. Water from higher lands is made to pass on, to the lower and thus the whole plot is supplied. This is called *field to field irrigation.*

In laying out a field channel, the following points should receive attention .—

(1) It should command all the area.

(2) It should be at one end of the field as far as possible so that the irrigator's land will not be unnecessarily divided.

(3) Different villages should have separate outlets and field channels.

(4) The number of outlets in the field channel should be minimum.

As water is supplied through field channels in turn a very large quantity is required, especially during the transplantation season, and hence their sizes should be so fixed, that they take only the required quantity.

6. Design of an irrigation canal:—The canal is mainly intended to irrigate the lands below it. Generally, the acreage under the channel is temporarily fixed, and the duty is also taken from the surrounding locality (as actually realised).

Then, Acreage Duty = Discharge in the channel = Q.

Regarding Q, i. e., the discharge, an allowance of 15–20% over the quantity, as arrived at by the above method, should be added towards the loss in seepage, percolation, evaporation, absorption and also possible errors in assumption.

$$\text{Also, } Q = AV,$$

where A is the area of cross section of the channel

and V is the mean velocity of the water in the channel.

The velocity in any channel is always fixed, taking into account, the soil through which the channel flows. There is a critical velocity for each soil which should not be exceeded. These velocities for different soils are noted below.

Critical Velocity

Light sand	1·5 to 2·0 ft. per sec.
Sandy clay	2·5 ,, ,, ,,
Soft clay	3·0 ,, ,, ,,
Hard clay or grit	4·0 ,, ,, ,,
Soft rock	5·0 to 8·0 ,, ,, ,,
Hard rock	8·0 to 12·0 ,, ,, ,,

Having fixed V for the particular soil, A is known.

Also, A = Average breadth × Depth, and D is also known by taking Kennedy's formula $V_c = CD^n$. Hence the width can also be known as the slopes given to a channel are about $\frac{1}{2}$: 1. The cross section of the channel thus arrived at may be verified with other formulae of Lacey and Bazin, and a final design fixed up.

Velocity and depth of flow :—The velocity usually given is as per Kennedy's silt theory. It is

$$V_0 = CD^n$$

With different values for D, the critical velocity V for the soil being known, the correct depth D is fixed.

The mean velocity allowable is X times V.

Velocity :—In ordinary soil the minimum mean velocity to prevent silting and growth of weeds is $1\frac{1}{2}$ ft. per second, and the maximum mean velocity to avoid scour is $2\frac{3}{4}$ ft. per second.

The velocity is generally increased near cross drainage works, such as aqueducts, to reduce the section of channel and minimise the cost of the cross drainage works. At these places a velocity of even 5 ft. per sec. is allowed.

Advantages of High Velocity :—(1) It reduces the time of transit of water and thus the transmission losses. (2) It reduces the cross sectional area of the canal, and hence the cost. (3) It retards the growth of weeds. (4) It lessens the deposit of silt.

Disadvantages of High Velocity :—(1) When there is erosion in the bed, it increases the cost of maintenance. (2) It lessens the command and prevents irrigation of high lands.

N. B.—It is advisable to maintain a constant velocity throughout the length of the canal, so that the silt suspended in it, may be carried on to the fields.

Width of berms :—The minimum width allowed for the berm of a channel is half the depth of F. S. D. so that the central line of the bank and the channel may be made parallel.

Berms are omitted (1) in small distributary channels running through very valuable lands, and

(2) where the channels run in full embankment.

The usual width of berm allowed is as follows:

For small channels :–d/2

For big channels :– $d/2 + 4$ ft.

The general rule for the width of berms is

$$\text{Width of berm} = C + (r_2 - r_1)\, d$$

where C = a constant;

d = actual depth of cutting at the section;

r_1 = horizontal component of slope of cutting in 1 ft. vertical.

r_2 = horizontal component of slope of bank in 1 ft. vertical.

Side slopes of channels :—A channel is generally excavated at 1 : 1 slope, but in course of time, due to silting up, it becomes $\frac{1}{2}$. 1 Hence while designing for the discharge of a channel, the slope assumed for calculations is $\frac{1}{2}$: 1 only.

Side slopes in canal excavation usually adopted for different kinds of soil are as follows .—

Side Slopes

	up to 8 ft. depth	More than 8′ depth up to 15 ft..
Light sand ...	2 or 3 to 1	3 or 4 to 1
Sandy loam ...	$1\frac{1}{2}$ to 1	2 to 1
Soft clay ...	1 to 1	$1\frac{1}{2}$ to 1
Hard clay or grit ...	$\frac{1}{2}$ or $\frac{1}{4}$ to 1	$\frac{3}{4}$ or 1 to 1
Soft rock ...	$\frac{1}{4}$ to 1	$\frac{1}{4}$ or $\frac{1}{4}$ to 1
Hard rock ...	$\frac{1}{8}$ to 1	$\frac{1}{8}$ or $\frac{1}{4}$ to 1
Embankment		
Inner slope ...	$1\frac{1}{2}$ to 1	2 to 1
Outer slope ...	1 to 1	$1\frac{1}{2}$ to 1

In alluvial soils, the sides of cutting are given up to ½ : 1 slope: hence, while calculating the discharging capacity of the canal, this slope is to be taken.

Curves:—For the easy flow of water in a channel, a minimum radius should be adopted. This minimum radius is generally taken as 20 *b*, where *b* is the bottom width of the channel.

The objection to a very sharp curve is that water has a tendency to erode the outside, so as to deposit silt on the inner side. This can be seen especially in soft and alluvial soils. Hence the radius of curvature should be large for channels in alluvial soil.

The greater the radius, the better is the curve for proper flow. But long curves offer difficulties in setting out (curves.) (In Northern India, the curves adopted are of a large radius, say 5000 ft. and more.)

7. Economical Section of a Canal. The section of a canal may be made economical in one of the following ways:—(A) The section may give maximum discharge for a minimum cross section; here the cost will be small. (B) The section may be made, such that there will be the least loss of water. This loss is mostly due to absorption.

(A) Channel with cross section for a maximum discharge:— (1) This form is obtained when the bed and the sides of the channel are tangential to a semi-circle, whose diameter is taken as the width of the channel at the full supply level. This section is one with minimum cross sectional area and the velocity is the maximum, as the Hydraulic mean depth here is the greatest (See Fig. 237)

CHANNEL WITH MAXIMUM DISCHARGE

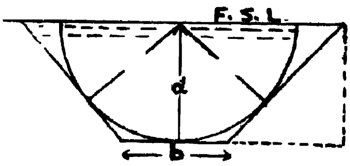


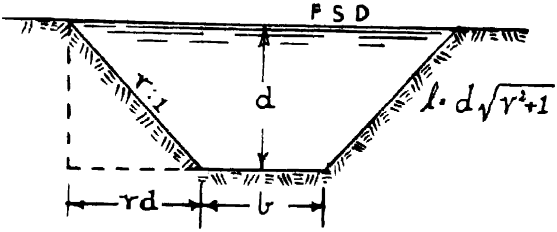
Fig. 237.

(2) There is another cross section for the maximum discharge as given by Sir T. Higham.

For any trapezoidal channel of area A, if r is the ratio of the horizontal to the vertical of the side slope, and s is the sum of the lengths of the two side slopes per unit of depth and it is equal to $2\sqrt{r^2+1}$ and M is the ratio of the bed width to the depth

$M = \frac{b}{d}$ then the hydraulic mean depth $= \left(\frac{\sqrt{M+r}}{M+s}\right)\sqrt{A}$. and this is maximum when $M = s - 2r$. (Fig. 238).

HIGHAM'S SECTION



$s = \frac{2l}{d} = 2\sqrt{r^2+1}$

Fig. 238

N. B. :—With a channel of slope $\frac{1}{2}$: 1 the maximum value of M works to about 1·236 or $1\frac{1}{4}$ roughly.

The discharge $Q = AV = AC\sqrt{mi} = AC\sqrt{A/P_w} \times \sqrt{i}$.

This is maximum when P_w is minimum. This is obtained for a rectangular cross section of the channel when $D = B/2$.

(3) For a trapezoidal section of the channel, the details of

MAXIMUM DISCHARGE FOR A TRAPEZOIDAL

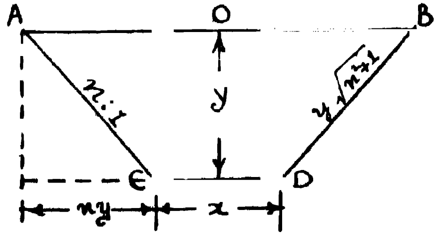


Fig. 239.

maximum discharge given with reference to Fig. 239 are given herewith.

If x is the bottom width, y is the depth of the channel and n/1 or n the side slope, then

$$x = 2y\,(\sqrt{n^2+1} - n).$$

(2) Hydraulic Mean Depth = $y/2$. (3) The three sides of the trapezoid touch the circle described with o as centre, and y as radius. (4) The top width is twice the side slope. (5) The sum of the top and bottom widths is equal to the wetted perimeter.

For the maximum discharge of the channel the depth of the channel is given by the formula

$$= \sqrt{2\sqrt{n^2 - 1} - n}$$

If two quantities in the formula are known, then the remaining one can be calculated.

This section is only economical for small channels. With large channels, the depth arrived at is great due to the extra lift involved and the work is not economical.

(B) Channel with cross section of least (absorption) loss of water :—The losses due to absorption are generally considered to be proportional to the wetted perimeter and the square of the depth, and to achieve these objects, the best section is one where B/D = 4. It is seen that when M is 4, there will be the least absorption.

Disadvantages of the economical section of a channel :—(1) As the depth is great in proportion to the width, there will be a heavy deposit of silt. (2) When the soil is porous, there will be a greater loss from absorption, due to depth.

8. Canal Excavation and Embankment :—For this purpose, the centre line should be aligned and set out, and then from the sanctioned cross sections, the profiles required. will have to be given at the cross sectional points and at other important places.

Embankment :—Before the beginning of the work, the top soil including roots, grass and other rubbish, will have to be removed from the surface, to receive the new work when an embankment is put. Necessary grips and trenches will also have to be excavated on the old surface.

Excavation :—If excavation is to be done for the channel, it is done vertically only to the bed width of the channel and the slopes are trimmed at a later date. This is to save the expenditure for the work in the slopes, as this work is easier. Dead men, (*Sakhis*) will have to be left at equi-distances to measure the actual depth

of cutting and to find the nature of the soil at different depths. The usual slope to be given in an excavation is 1 : 1.

The channel should be excavated correct to the bed level and to the width as sanctioned. The stuff removed from the excavation is to be deposited as a spoil bank, away from the edge of the channel, leaving a good width of berm. The spoil bank should be of uniform height throughout, and when required it may be only broadened. (See Fig. 240).

Balancing depth of cutting :—Whan the spoil from the excavation of a canal is equal to that required to form the banks,

CHANNEL SECTION IN FULL CUTTING

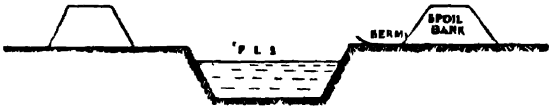


Fig. 240.

as per sanctioned standard design, the canal is said to be in *balancing depth of cutting.*

Equation for Balancing depth of cutting :—D for a canal with side slope of cutting 1 : 1 with two banks of equal cross section and $1\frac{1}{2}$: 1 slope is $D^2 - (3h + B + \frac{1}{2}b)\,D = -h\,(B + \frac{3}{2}h)$ where h is the vertical height of the top of the bank above the bed of the canal.

b is the bed width of the canal

B is the top width of the bank

and D is the balancing depth of cutting (See Fig. 241).

SECTION SHOWING BALANCING DEPTH OF CUTTING

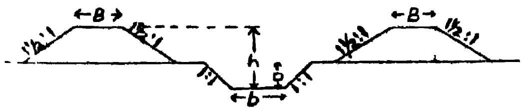


Fig. 241.

Partial cutting and embankment :—The channel is sometimes to be excavated in partial cutting (See Fig. 242). The bank

PARTIAL CUTTING

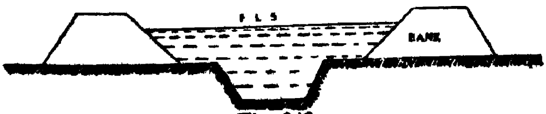


Fig. 242.

portion should be of selected material and a core wall should be provided in the centre. This core wall should be covered with proper material to prevent it from cracking.

Full Embankment:—Near aqueducts or other cross drainage works, high embankments are necessary for the channel. The work in this portion should be done very carefully. The soil for the embankment should be specially selected and mixed with other soils if necessary, to get a compact mixture, which should be as far as possible impervious. (Black soil should never be used for an embankment). The soil selected shall be laid in even layers of 6 in. adding water whenever necessary and well compacted by roller, (1·3 tons Dentated Roller). The whole embankment should be raised evenly, so that there may be no slips at a later date. (For an embankment the slope given should be $1\frac{1}{2} : 1$. As said already in chapter on Earthen Dams, the borrow pits should be properly attended to). The core wall in the centre should consist of good clay puddle and should be brought to the full supply level or even higher. See Figs. 243–244.

FULL EMBANKMENT

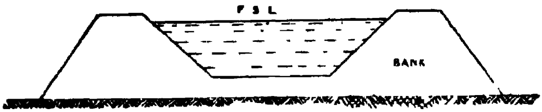


Fig. 243.

HIGH EMBANKMENT

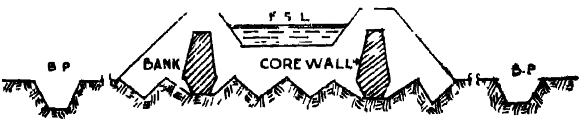


Fig. 244.

Every day after the work is over, to prevent the top surface of water being evaporated, either a pond of water is provided at the top or the surface is covered with wet sand temporarily to be removed the next day when the work begins. The whole embankment should be well consolidated at the optimum moisture content. After rolling, the embankment should be tested for density.

9. Maintenance of channel :—Just as in an earthen dam or embankment, repairs are necessary occasionally for the channel embankment also. These are repairing a leak, filling up a breach, or repairing a scour, etc. All these will have to be done similar to the earthen embankment. The additional work required for a channel is the clearing of silt and weeds in the bed of the channel, which impede or obstruct the flow in the channel. This is done every year when the channel is shut during the off season, and the necessary charges are borne either under the acreage cess fund or the irrigation cess fund.

The repairs to masonry, revetment, etc., will have to be done also occasionally and this is done at government cost.

10. Aquatic Weeds :—Plants which grow abundantly even under water are called *Aquatic weeds* or water-loving plants. Aquatic plants are a great menace to agriculture. They spread throughout the irrigation and drainage systems, along canals, ponds, etc.

The disadvantages of aquatic plants are

(1) They impede the flow of water in the canals and the drainage systems; and sufficient water cannot be supplied to the lands, or the drainage system fails. (2) They render the water of the ponds and streams dirty and create hindrance to swimming and fishing. (3) Evaporation losses in the canal are increased. (4) They supply excess of organic matter to the water, and it is harmful to use such water for domestic purposes.

The spreading of aquatic seeds is due partly to birds which eat the weeds and excrete the seeds. Wind is also partly responsible for this spreading.

Types of weeds :—There are different types of aquatic weeds. They are

(1) Weeds which grow completely submerged under water. These are water-weeds, horned pond weeds, and horsetail, which grow usually in ponds. (2) Weeds which grow on the water line. These are weeds like reeds, water-plantains, bull-rush grass, cat's-tail, arrow-head, etc. (3) Weeds which grow on the suface of water. They are usually weeds like water-lettuce, spiked grass, thorny fruit-weeds, etc.

Factors which may increase or reduce the growth of weeds are

(1) *Temperature* :—Temperature of about 20° to 30° is most suitable for the growth of aquatic weeds.

(2) *Velocity* :—Weeds-growth is reduced where the velocity is greater than 2 ft./sec.

(3) *Depth of water* :—Aquatic weeds grow even up to 18 ft. below the water surface.

(4) *Deposit of silt* :—Silting of the waterways increases the growth of aquatic plants.

(5) *Reduction of light due to turbidity* :—The growth of aquatic weeds is hindered, if light is reduced due to the turbidity of water.

Control of Aquatic plant growth :—The growth of aquatic weeds can be controlled by the following methods :—

(1) Rush rotation of crops. (2) Mechanical. methods. There are different types of mechanical methods for the control of weeds. They are (a) Dredging, (b) Chaining, (c) Drying, (d) Hand clearing, (e) Burning.

(3) Chemical methods :—The chemical methods are :— (a) Contact spraying, (b) Application of Penoclor, and (c) Biological method.

11. Regulation Works :—The surface level of the canal fluctuates with the supply in the river and the demand for irrigation, and works are required to control the velocity of flow, to maintain the required water levels and effect proper distribution throughout the canal system. These are called regulation works. The important regulation works are :—

(1) Regulators. (2) Falls or Drops (in North India and South India). (3) Rapids. (4) Escapes. (5) Regulating notches. (6) Dividing dams. (7) Irrigation sluices. (8) Pipe outlets. (9) Gauges.

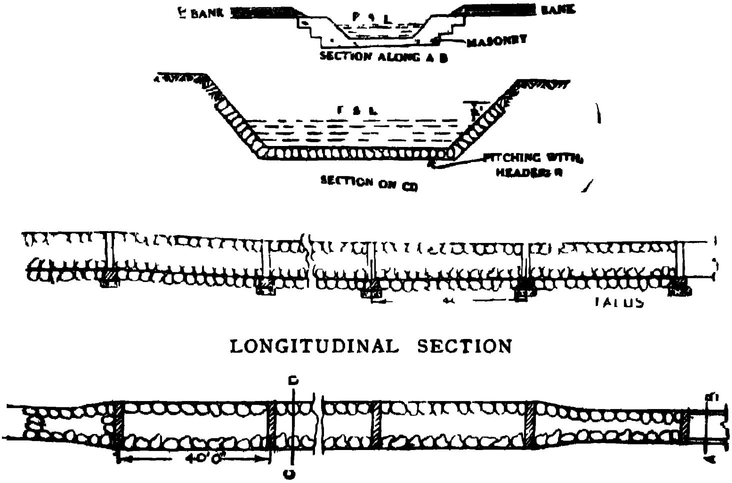
1. *Regulators* :—These are structures across a canal which have their floor at bed level and are provided with piers with grooves, for the use of shutters to regulate the water level and thus distribute the total discharge, to the canals below, in the required proportion.

2. *Falls or Drops* :—The works by which the bed of a canal is led down from a higher to a lower level are called Falls.

If the slope of the country exceeds the slope which can be given to a canal, falls or drops are constructed. Vide also para on design of drops lower down in the chapter.

3. *Rapids* :—Instead of falls, rapids are sometimes employed to get the necessary change of level. The slope usually employed in rapids is about 1 in 15. The floor is paved with boulders grouted with mortar, and necessary binding walls are constructed at intervals of about 40 ft. and the sides are also protected. A *Talus* at the foot of the rapid is a necessity. Vide Fig. 245. The

RAPID



Plan
Fig. 245.

best form of a rapid is that in which the slope is reduced as it descends, so as to gradually assimilate with the bed of the canal. See Fig. 246.

LONGITUDINAL SECTION OF A RAPID

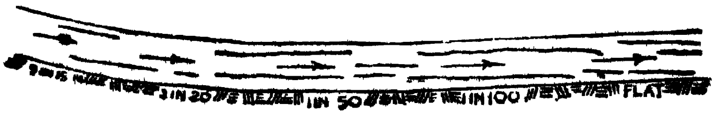


Fig. 246.

Where used :—When there is a considerable fall and a sudden drop of the country and the ground is hard, a rapid is provided.

Construction. (1) The sides and the bottom should be pitched against any erosion. The stones for pitching should be used only as headers.

(2) If the canal is large it may be necessary to build curtain walls at intervals.

(3) The bed width of the channel in a rapid should be made wider, to lessen the depth or flow and thus the scouring. The pitching should be made rough to decrease the velocity.

(4) The bed and sides should be protected for some distance both upstream and downstream, to destroy the extra velocity. The best material for the rapids is stone boulders.

4. *Escapes* :—Escapes are provided where the surplus water can be easily got rid of, when there is the necessity for the control of flow of water in the canal. They should be avoided as much as possible, because they

(i) Obstruct the irrigation purposes, (ii) waste the water in the canal and (iii) cause irregular regulation of water.

Escapes should be located as near to the drainage lines as possible, since this helps in reducing the length of the lead-off channel to be excavated. To overcome the difference of levels between the canal and the river, to which the channel tails, falls may have sometimes to be constructed in its course.

Surplus Escapes :—Surplus escapes are generally in the form of weirs, flush escapes or sluices. They may be built as separate structures or may be combined with the outlets. Surplus escapes are so constructed that they discharge all the water, down to the levels of their sills.

Sometimes escapes are built with a bridged overfall weir divided into a convenient number of bays. Sufficient width should be provided for the bridge, since it may be used for traffic also.

Escapes can be located with advantage near aqueducts or drainage crossing, since they can be combined with these works. A canal escape can also be used as a sluiceway, for scouring the silt from the parent canal.

Tail Escape :—An escape constructed at the tail end of an irrigation channel which ends in a natural drainage is called a *Tail Escape*. The object of providing such an escape is to maintain the full supply level required at the tail end of the channel, and to enable the excess of water to get surplused without causing a breach.

The general type of a tail escape is a weir constructed across the channel with its sill at F. S. L. of the channel. This allows only the surplus water to pass. At the bed level of the canal, a few sluices are provided in the centre of the body wall of the weir. Vide Fig. 247. These sluices are generally made use of to

LONGITUDINAL SECTION OF TAIL ESCAPE

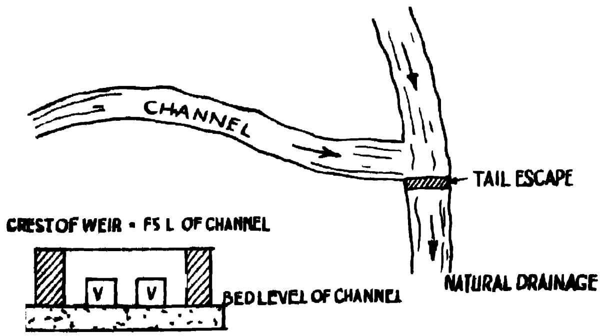


Fig 247.

scour the silt deposited at the tail end of the channel. The tail end should be cross-bunded, and the top of the bund should be below the ordinary level of the channel bank, thus forming a breaching section or an emergency spillway, when a tail escape is not provided.

5. *Regulating Notches or Dams.* Where changes in the depth of channels are made without any drop in the bed level, trapezoidal notches built on a floor across the channel, are frequently constructed to regulate the supply. Such notches are not fitted with planks or shutters but automatically regulate the discharge above and below the work. These are termed regulating notches.

6. *Dividing Dams.* Where the channel is bifurcated and water is to be divided proportionate to the areas under the channel, a Dividing Dam is built. This automatically subdivides the flow in the required proportion, as the lengths of the notches constructed are proportionate to the areas to be irrigated. The ryots are in favour of this method, as it is easily understood by them (See Fig. 248.)

DIVIDING DAM

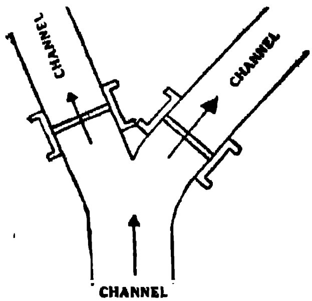


Fig. 248.

7. *Irrigation Sluices.* This is a structure built to allow the required discharge from the channel to the lands. This is similar to a tank sluice. A culvert or pipe is used to carry the required discharge. At each end of the culvert suitable wings are provided, to retain the earth slopes and so form effective bank connections, and also to serve as stop-walls to prevent percolation. The head wall is fitted with a shutter running in grooves, built in masonry, by means of which, the vent can be opened or closed or the amount of opening regulated for conveying the water of the parent channel through one or more culverts or pipes under the canal banks into the branch distributary or field channel.

Each sluice should pass its fair share of bed silt and so the sills are built at the bed level of the parent channel.

8. *Pipe Outlets* :—These are structures provided to supply water to the fields from the distributary channel and are generally provided with pipes for their barrel or tunnel portion. These pipes were originally of cement, glazed earthen-ware or stone-ware, and now they are mostly of cast iron (after the factory at Bhadravati in Mysore was established). They have a front and rear head-wall of sufficient length, to retain the earth slopes. These sluices are called Pipe sluices or *Madavoys*.

The front head wall should be located at a distance equal to half the supply depth from the intersection of the channel bed and side slope and the top of the head wall should be at full supply level.

PIPE OUTLET

SECTION

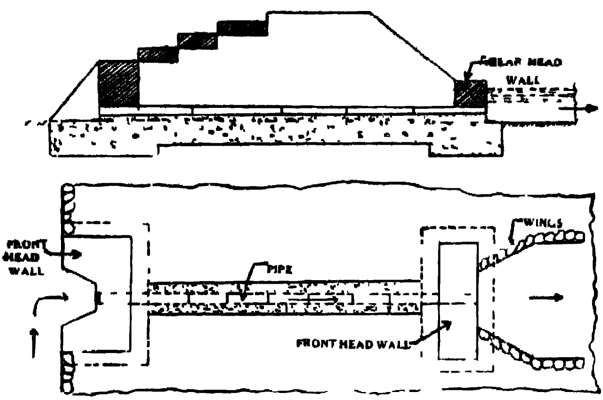


Fig. 249.

There are two types of head wall, one with wing walls and the other without them. (See Figs. 249–250)

SKETCH PLAN SHOWING HEAD WALL WITHOUT MASONRY WINGS, BUT WITH ROUGH-STONE REVETMENT

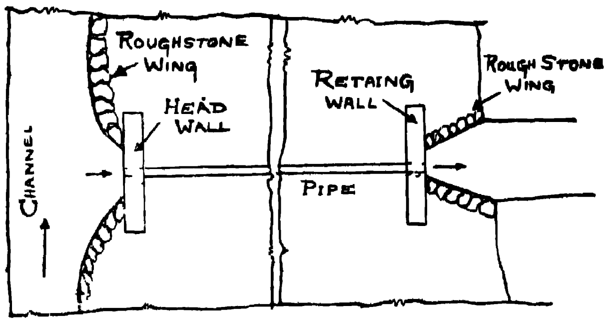


Fig. 250.

The discharge varies with the silt accumulated at the rear bed, water level in the field channel, and the level of the field irrigated; hence all pipe outlets should have free outfalls, to get the maximum discharge. The minimum diameter of any pipe-outlet used is 3 in. The discharge is therefore disproportionately high in

many places and hence attempts should be made to use pipes of 6 in. diameter and more, and group all areas to minimise wastage. A layer of concrete is provided below the pipe to prevent settlement. The level of the pipe is kept at least ½ ft. above the level of the highest land commanded under the pipe.

Usually lands of the same village are irrigated under the same pipe. It is also economical to reduce the number of outlets to the minimum, to prevent wastage of water. For calculation of discharge, as the depth of water in the parent channel is varying, 2/3 of the full supply discharge or sometimes even ½ the discharge is taken for the calculation of the diameter of the pipe. The formula generally used is

$$Q = \left[0{\cdot}0438\, d^2 \times \sqrt{\frac{d}{0{\cdot}36\ (4{\cdot}17\, d + L)}}\right] \times \sqrt{h}$$

where, Q = discharge in cusecs

d = diam. of pipe in inches, L length of pipe in ft. h head in ft.

N. B. μ is taken as 0·0075 for pipe (earthern-ware pipe), μ is the coefficient of fluid friction of the pipe, $\mu = 0{\cdot}005$ for clean iron pipe and 0·01 for slightly encrusted iron pipe.

9. *Gauges*:–These are intended to record the variations in the depth of the water-supply in channel, and to regulate it also. They are fixed at the heads of all branch channels and regulators and near escapes. They are also fixed on the piers of bridges and on the wing walls of irrigation sluices or outlets.

10. *Tail Reservoir*:—To construct the tail reservoir, the natural contours of the ground should be favourable and a proper site should be selected. Tail reservoirs are reservoirs built at the tail end of the irrigation channel, to store all the waste water. This type of reservoir is only built when the channel used for irrigation is large and where much water is being wasted, so that it may be supplied to the lands (at the tail end of the channel), where usually the supply of water is not satisfactory.

11. Drops or Falls:—The drop is a retaining wall and to protect it, apron and wings are constructed. The mean velocity of water passing over the crest of the drop-wall across the channel is greater than the velocity of water in the channel above. It is

therefore desirable to contact the waterway over the drop in such a way that the surface flow and the velocity of the canal water are not affected by it. This is done by providing a rectangular notch over the weir crest.

For the protection against erosion, the following devices are adopted:—

(1) The impact of water on the bottom of the canal at the drop requires some sort of paving.

(2) The increased velocity generated by the fall, persists for some distance down in the canal and erodes its banks. Hence a stilling pool or water-cushion is provided at the foot of the drop, with a larger section than the regular area of the channel, so that its energy may be dissipated.

(3) Waves are produced by the commotion of the falling water and they persist for some distance down the canal; hence a paving—concrete lining or stone in cement mortar—and banks should be provided (for some distance downstream).

12. Design of drops :—The drop should be vertical on a flat apron or water cushion. The upstream face should be made water-tight by the provision of puddle clay covered over with stone pitching.

The full discharge of the channel should be allowed through trapezoidal notches. The sill level of the notches should be at the bed level of the channel. The drop wall should be designed as a retaining wall with base width $b = \dfrac{H + d}{\sqrt{\rho}}$ (1)

where ρ is the specific gravity of the material of the wall.

DESIGN OF DROPS

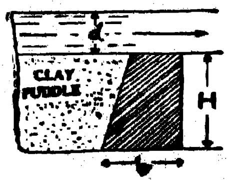


Fig. 251 (1).

The length of the wall = not less than $\frac{7}{8}$ of the bed width of the canal upstream, i. e. less than full width of the canal. (2)

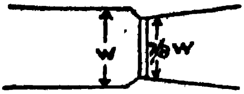


Fig. 251 (2)*.

The width of the piers of the notches at F. S. L.
$= $ not less than $\frac{1}{2}\, d$

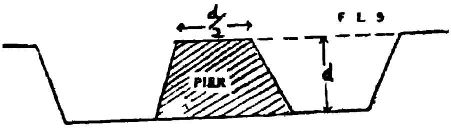


Fig. 251 (3).

and the top level of pier should be at F. S. L.
except when a bridge is provided. At the two abutments only ½ piers are constructed.

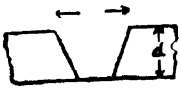


Fig. 251 (4).

The top width of the notch not to exceed $0{\cdot}75\, d$, if notch is free, and $= d$, when notch is submerged. (4)

The thickness of the drop wall at sill level
a should be between $(\frac{1}{2}d + 1)$ and $(\frac{1}{2}d + \frac{1}{2})$ (5)

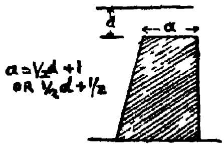


Fig. 251 (5).

The width of the main apron at drop,

$$A = L + \tfrac{1}{2}d \qquad (6)$$

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* The number in bracket against Figs. 251 (1) to 251 (14) for example, 251 (1) and 251 (2), refer to the number of the formulae given.

and the minimum should be the width of the canal downstream.

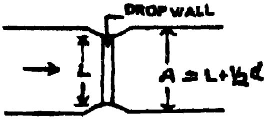


Fig. 251 (6).

x = The length of the solid apron $= 2d_c + \sqrt{d_c h}$ (7)
with a minimum of $4 + \sqrt{d_c h}$

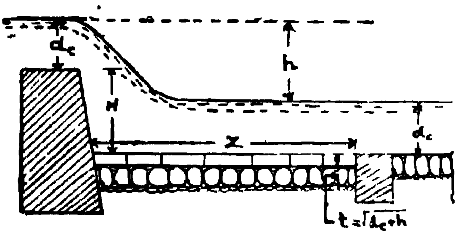


Fig. 251 (7) & (10).

Water cushion :—If a water cushion is provided the depth of water cushion is given by,

Dyas as, ... $(X + d_1) = d_1 + d_c^{1/3} h^{1/2}$

In Punjab ... $X = d_c^{1/3} h^{1/2}$

Madras ... $X + d_1 = \frac{1}{2} d_c \sqrt{h}$ (8)

d_1 is the depth of Water in the Channel below.

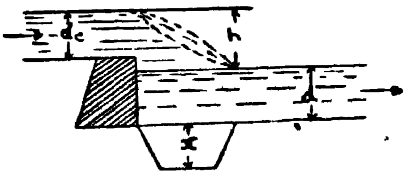


Fig. 251 (8).

If $d_1 = d_c$, then X becomes zero or negative up to 4′ — i. e. under these conditions, a solid apron should be constructed.

Length of the horizontal floor of the cistern of the water cushion, and a minimum should be $2 + 2\sqrt{d_c h}$ (9)

The downstream side, beyond the horizontal portion, should be built up to a slope of 1 in 5, if pitched with dry stone masonry or

1 in 3 if with solid masonry. This is to easily carry away debris from the cistern. (9a).

The thickness of the masonry apron depends upon the nature of the foundation. It should be thick enough to resist the uplift.

i. e. not less than $\sqrt{d_c h}$ (10)

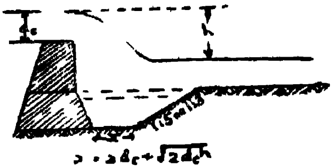


Fig. 251 (9 a).

The length of revetment (upstream side.)

Length of revetment = $3d$ or 10 ft. min. (11)

Downstream revetment = $(4d+h)$ or 20 ft. minimum (12)

Length of bed pitching = $\frac{1}{2}$ length of revetment. (13)

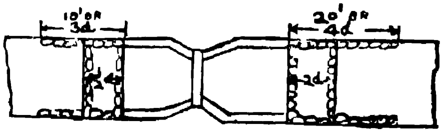


Fig. 251 (11) (12) & (13).

Connection of drop wall with the bank. This is done by the construction of either (a) wing wall or (b) core wall.

The wing wall is the better of the two. It requires less repairs and it is good for high falls. It is used when the width of the channel is great and the discharge also great (say 100 cusecs and more). The core wall is cheaper and adopted when rough stone is available in plenty, and for small drops.

Discharge through trapezoidal notch.

If the notch has a free overfall,

$$Q = 5{\cdot}35\, C\, d^{3/2} (l + 0{\cdot}4\, d^{n}).$$

If the notch is submerged,

$$Q = 5{\cdot}35\, C \sqrt{d - E}\, \left\{ \left(\tfrac{2}{3} E + d\right) l + \left[\tfrac{4}{15} E^2 + (d - E) E + 0{\cdot}4 (d - E)^2\right] n \right\} \quad (14)$$

In the above equation,

(1) 5·35 is substituted for $\frac{2}{3}\sqrt{2g}$,

(2) C is generally taken as 0·70,

(3) Velocity of approach is neglected.

where Q = Discharge through rectangular portion + discharge through the sloping or triangular portion,

E = Submersion = depth of tail water over the sill of notch,

l = width of the notch in feet,

c = coefficient of discharge,

d = depth of flow F. S. L.

n = 2 tan α, where α is the angle the side of notch makes with vertical. (14)

For problems on Notches and Drops, refer to the Book *Solutions of Problems in Irrigation* by the authors.

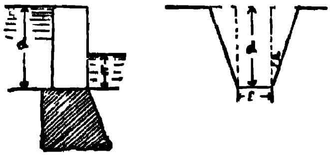


Fig. 251 (14).

13. Canal Lining:—Lining is a permanent improvement for a channel especially if the soil is pervious. There are different methods of lining a channel :—

(1) *Cement Mortar Lining* is generally used in conjunction

LINING OF CHANNEL

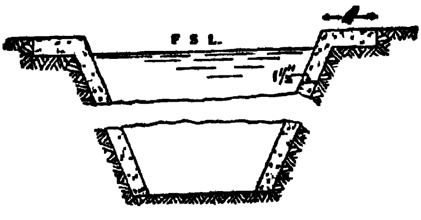


Fig. 252.

with some of the protective material. A coat of cement mortar 1 : 4 ratio and 1½ in. thick is applied to the slope of the channel

which should be formed beforehand and the surface of the slope also well consolidated. The mortar is applied and the section of the channel formed as shown in Fig. 252.

(2) *Concrete lining*:—The proportion allowed is 1:3.5 or 1:3:6. The thickness of the concrete lining is about 4 in. Expansion joints are left at about every 50 ft. apart to a width of ¾ inch. A thin coat of cement plaster 1:3 is applied to give a smooth surface. In the section so lined, N = 0·015; and it is seen that there is a saving of 40 per cent of the discharge in the channel. In ordinary soil, the lining is given a slope of 1½ to 1 and in sandy soil 2 to 1. The thickness of the lining depends upon the angle of repose and the surcharge.

(3) Lining with hydraulic lime concrete 4 in. thick and hydraulic plaster is done in the same way as above. A velocity of even 6 ft. is allowed in this section safely.

(4) *Brick lining*:—This is done with either one or two bricks thick. With one layer, it is done with brick on edge, with 2 layers good cement should be used between the two bricks. The advantages of brick lining are (i) cost is low, (ii) no costly equipment is necessary, (iii) no cracks are formed due to expansion or contraction, (iv) repairs can be carried out expeditiously.

The slope of the channel is prepared and consolidated before the brick is placed in position. For this work, good, well-burnt bricks should be used, as otherwise, owing to the moisture present, the bricks may give way. In good many instances, the bricks are only pointed with good cement mortar, and in some cases they are plastered with cement.

(5) The other kind of lining is of rubble stone with an average thickness of 6 to 9 inches, well grouted with concrete, and sometimes cement plastered. Wherever the cement concrete work is done for the lining, it should be so done, that all the voids are completly filled up, because watertightness is the important item in lining work.

The lining work saves 60% of the waste of water which would occur if the channel were not lined.

(6) Clay puddle is used as a lining to prevent percolation. This should be kept moist and covered. It is said that this lining reduces seepage by [illegible]0%.

(7) Road oils and Sodium carbonate lining are sometimes used and they are found to be not satisfactory.

General—For the expansion joints in cement concrete work asphalt and puddle are used. The asphalt melts in high temperature and protrudes outside, and the velocity in the channel is thus checked.

The expansion joints are filled with tar and bitumen. This is more effective, but does not last long.

In unlined channels, the seepage losses are estimated at 10 cusecs per million square feet of wetted area and if lined, it is reduced to 3 to 4 cusecs.

Advantages of Lining:—(1) There is saving of water to be let into the canal, as there is decrease in the conveyance losses. (2) It prevents the growth of weeds and moss. (3) The maintenance cost is reduced. (4) It reduces the friction as the surface is made smooth, and as such, the velocity can be increased and the section of the channel reduced. (5) It prevents the rise of the water table nearby, and thus the waterlogging. (6) It provides safety to the bank of the channel against breaching, especially in side-hill slopes (by prevanting leaks). (7) It decreases or prevents the erosion of the sides or the bed of the channel, due to the higher velocity produced by lining. (8) By preventing waterlogging, it increases the fertility of the soil.

Lining with Concrete and Bamboo Reinforcement :—This method was tried by the author for a length of about 1000 ft. for a distributary channel and was found to be very satisfactory and economical. The work was done about 17 years back and is standing very well. A note is given in the Appendix of the author's book-Solution to Problems in Irrigation Engineering.

SILT

1. **Introduction** :—Silt is of two classes :—

(a) Bed silt which is also called the Dragged or Rolled silt,

(b) Suspended silt.

The nature of the silt depends upon

(i) The topography of the country, and (ii) Rainfall..

N. B.—In South India, bed-silt consists of coarse shingle and heavy pebbles; and suspended silt consists of finer sand and earthy argillaceous matter.

Causes for the Silting up of Channels :—1. A channel will silt, if the silt load is in excess of the discharge which is capable of carrying it or if the silt grade is high.

2. When the channel section is not of regime, that is, either the slope is not sufficient or the B/D ratio is not properly designed.

3. When the discharge in the channel is not constant, but varying; because, if the discharge is small and spread over a long stretch, the channel will generally silt.

4. When the head sluice is not properly designed, and there is turbulance at the entry, which causes the draw of a higher charge and a higher grade of silt.

5. When the outlets are not properly designed to draw the silt from the parent channel to the distributary or the field channel.

6. And finally, if the maintenance work of the channel is not properly and promptly attended to.

" Silt is both a bane and a blessing to the cultivator. "

The bane or disadvantage of silting is due to the following reasons.

1. The channel, even with precautions, gets silted up, especially in its first reaches and thus the discharge in the channel is reduced. As a result of this, when water is most urgently required, the cultivators suffer, especially those at the tail end.

2. The cost of removal of silt is very heavy and this removal should be prevented as much as possible. That is, arrangements should be made that the silt does not enter the channel.

3. While removing the silt, the channel has to be closed for some time and this means again loss of valuable crops.

Blessing or Advantages of silt :—

1. Silt is a good fertilising agent.

2. The silt transported in the channel adheres to the sides and the top surface of the channel in a cutting, and berms are formed, which means a good savings in the cost of forming berms.

3. It reduces seepage by forming a good lining at the sides and the bed of the channel.

4. It retards the growth of weeds, because the silt-laden water is turbid, and it prevents sun's rays reaching the bed of the channel.

If silt enters these, to remove it, all possible means such as flooding it out, etc. should be done; and when these are not satisfactory, either mechanical or manual labour should be used.

8. Silt in Reservoirs:—All the silt from the catchment area of a reservoir enters a reservoir and in many cases the capacity of the reservoir is reduced very soon.

The heavier silt in a reservoir bed will be where the valley joins the reservoir, and the finer silt is carried very near the bund i.e., the deepest portion.

The rate at which a reservoir gets silted up, depends very greatly on the proportion which the run-off bears to the capacity of the reservoir.

The silting of some reservoirs is noted below:—

Name of Tank	Catchment area in sq. miles	No. of years	Percentage of silting
Bombay			
Muchkundi Tank	26	41	16
Gokak reservoir	1080	33	23
Dambal (Dharwar)	43	48	18
Mavinkop	6·25	46	17
Lake Fife		22	18

N. B. The Dambal was only half silted and Gokak was being silted up very slowly.

Silting of tanks cannot be generally prevented easily; and by silting, the capacity of storage is reduced, and the irrigation under the tank suffers. The only remedy now being adopted to increase the capacity or restore the original capacity of the tank is by raising the level of the crest and consequent raising of the top of bund, etc. As this is very expensive, special precautions should be taken to *prevent silting up of the reservoir.*

(1) In big reservoirs, the silt deposited is reduced by introducing scouring sluices. For example, the Assuan dam across the river Nile in Egypt has 180 sluices and most of the silt is scoured away. Bhatgarh or Lake Whiting in Bombay is not affected by silt.

To prevent the silt from entering the reservoir, the system of serial reservoirs (serial tanks) is recommended. Here the uppermost reservoir gets silted up and the other reservoirs are left free.

(2) Another method employed to prevent silt entering the reservoir is to encourage the growth of vegetation in the catchment area which will arrest the flow of silt. Plants such as Honge and Aloe are planted in the drainage area at distances apart accross the natural flow,

9. Silt at the head of the regulator in the pocket of the scouring sluice :—This is prevented from entering into canal by

(1) *Still Pond System* :—A pocket or pond is created in front of the scouring sluice, between the head regulator and the dividing wall. This is made wider than the approach channel and its bed is lowered by 2 to 3 ft., below the sill of the channel. The supply required for the canal enters the pocket and the velocity of water in the pocket is very much reduced on account of its excessive waterway. The silt deposits in the pocket and only clear water enters the canal.

The canal regulator is provided with a high crest wall. When the silt in the pocket has accumulated to say 2 ft. below the sill level of the regulator, the canal is closed and the under-sluice gates are opened to scour the silt deposits in the pocket. The high velocity generated will scour the channel immediately in front of the entrance to the canal, and thus furnish capacity for subsequent use as settling basin to be again sluiced out.

To prevent silt entering the canal, the sluice shutters are built in tiers so as to enable the canal to be fed from the water drawn from near the surface of the river.

The entrance to the canal is controlled by Flash boards of some similar device that the water will flow into the canal, as a thin film from the surface only, leaving the sediment at the bottom,

(2) *Providing scouring sluices or under sluices in the main weir wall.* When the scouring sluices are open, the head regulators should be kept closed. This is to prevent admitting the disturbed silt-charged water into the channel. When the head regulator is opened, the gates or the scouring sluices should be kept closed, to reduce the velocity of approach and thus deposit the silt in the pocket itself. The gates of the head regulator should be

arranged to be opened from the top and not from the bottom, so that the lowest point of the draught is as high above the head as possible.

(3) The waterway of the head regulator should be greater than the area of the approach channel to reduce the velocity at the intake.

(4) When the water is very turbid, only a small discharge should be admitted into the canal to reduce the velocity.

(5) Only clear water should be admitted into the canal from the river. This is arranged by having the sill of the head regulator at a higher level than the bed of the approach channel.

Silt at the head of a Regulator is also removed by excavating a new approach channel, as was done in the case of Sukkar canals. Vide Fig. 253 (c). Fig. taken from Bhagirath Magazine.

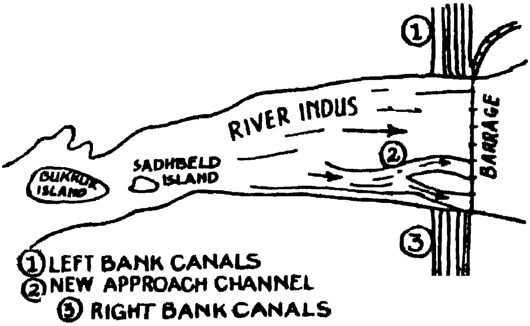


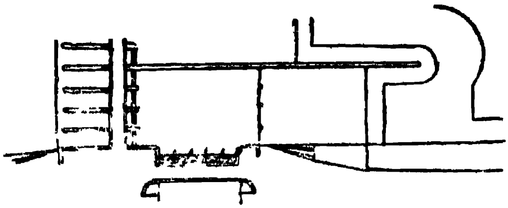
Fig. 253 (c).

Recently many of the modern canals are provided with silt excluders in the pocket of the scouring sluice just above the head regulator.

The **silt excluders** are devices by which silt is extracted from the water entering the canal or precluded from entering it. They are used at the head of the main regulator and also at all branch and distributary regulators. These consist of tunnels 6 to 8 ft. wide and covered on their top. The top of the slab is at the sill level of the head regulator. These extract silt from the water and lead it to a natural drainage, that is, the water is made to pass to the river through the under sluices. The bed is divided into a number of portions or compartments each of which is allotted to

one tunnel portion. These compartments are divided by piers to prevent cross currents and eddies. The water above the slab is free from silt and enters the canal. These piers support the roof on the top. (See Fig. 254).

SILT EXCLUDER



PLAN

SECTION

Fig. 254.

The silt excluders are found to be working more efficienth than other systems in use until now.

10. Silt in the Canal :—Silt if allowed into the canal causes much annoyance and expense. Instances are not rare, where the silt etc., carried into the canal during high floods, so depleted its capacity, that it could not carry the water needed for irrigation, and it became necessary to close the canal and clean it during the height of irrigation season, at great expense and to the great injury to the crops. Hence measures should be adopted to prevent the entrance of silt and sand into the canal.

Silt must be prevented as far as possible from entering into the canal, but as it is impracticable to do so, measures should be adopted to remove the same from the canal. The methods adopted are

(a) Flushing the canal, (b) Provision of silt escapes (c) Provision of silt traps.

(a) Flushing the Canal :—In this method, the silt in the canal is washed by sending a large quantity of water more than the normal, for the purpose of flushing. If water of low turbidity is allowed into the canal, it can easily remove the bed silt from it.

This method is not quite practicable during dry weather, as the necessary abundant supply of water may not then be available.

(b) **Silt Escapes** :—Generally a large quantity of silt is deposited in the first reaches (say 5 to 10 miles) of a channel. Along this length, wherever convenient to lead the discharge into the natural drainage, silt escapes are provided. The silt escape is only a weir wall with its top at F. S. L. provided with vents and with gates, having the sill of the opening of the vents about 1 or 2 ft. below the bed level of the channel. These vents are opened often, during the rainy season and the silt depleted.

(c) **Silt Traps** :—These are constructed to prevent the silt from entering into the canal. At the inlets (natural drainages entering the canal) pits or basins are formed to receive a large quantity of silt carried down from the drainage area above. The sides of this basin (or pits) are protected with either masonry or rough stone. The quantity of silt collected is removed, as often as practicable.

The quantity of silt received is studied, and the capacity of the basin adjusted, so that this removal may be effected not more than 3 times in the rainy season.

The silt trap is located just a few feet from its junction with the natural stream and on the upper side of the channel, so that there may be free discharge of the drainage water into the canal. Protective works should be constructed at this place. See Fig. 255.

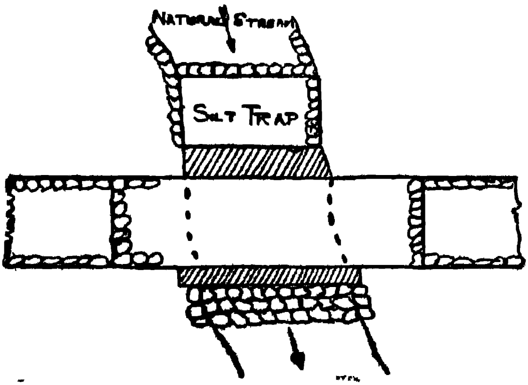


Fig. 255.

Silt traps are also erected in the bed of the channel itself. This is done in the same way as the above, and this is adopted so that the regime of the channel may not be affected by the silting up of the bed. See Fig. 256.

N. B.—The first reaches in a canal should be given a more longitudinal slope to produce a higher velocity so that the silt may be easily carried down.

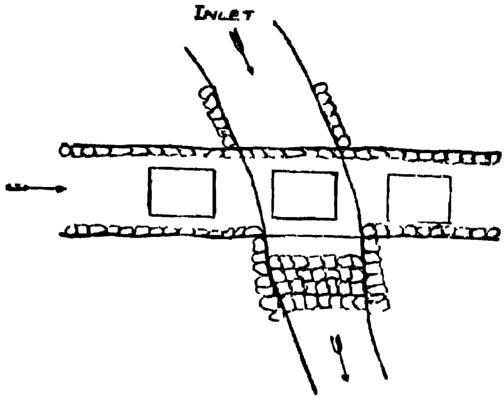


Fig. 256.

Other methods employed are:—(1) Silt Ejector and (2) Silt Vane.

The *Silt Ejector* is a contrivance by which the silt, after it has entered the canal, is extracted or removed out of the canal. It consists of a number of piers, 2 to 3 ft. high, for the full width of the main canal. It is covered with an R. C. C. slab to form an under tunnel. The bed silt in the main canal passes through them down into the canal. The section of the canal is made bigger just above an ejector, to reduce the velocity and drop the silt at the place. This is done by lowering the bed level of the canal (which helps in receiving the silt). Two or three such sets are provided, one below the other, in the first reaches of the canal, and at the end, a cross regulator is constructed with a weir at the downstream side of the canal. A lead off channel is taken to the

SILT EJECTOR

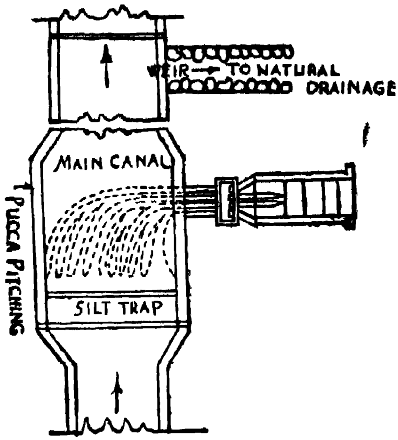


Fig 257.

river and gates are provided at the exit for proper regulation. (See Fig. 257).

These ejectors are more efficient than the usual silt traps and scouring sluices originally adopted in the canals, and are replacing them.

SILT VANE

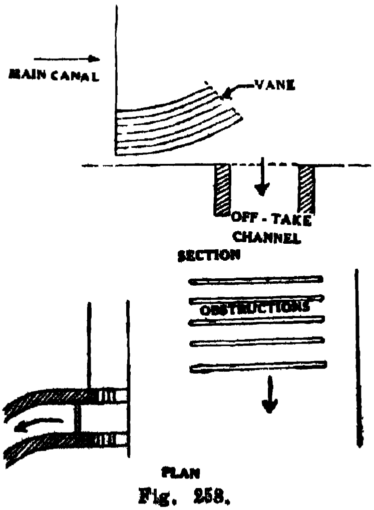


Fig. 258.

Silt Vanes :—These are constructed in the main canal at the head of a distributary or a branch. They are walls or obstructions, to obstruct or to divert the current to exclude silt entering the smaller channel. These obstructions or low walls should be so constructed that the lower layers of water should not enter the off-take.

The vanes are either straight or curved to the flow. The curved ones are used

SILT VANE

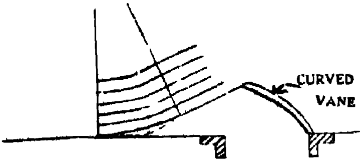


Fig. 259.

when the discharge of the off-take is more than one-third of the main canal, and when the discharge is less, straight vanes are used. (See Figs. 258–259).

Comparison between Excluders and Ejectors.

Excluders.	Ejectors.
1. This is a costly process as the work is heavy being subject to the action of the river.	1. The total cost is less, as the work is light.
2. The work is concentrated at only one place.	2. Separate outfall works are necessary.
3. As the channels are large they are not liable to be easily blocked up.	3. The orifices being small, they are likely to be choked up by the debris, and hence a trash rack is required.
4. Working head required can always be made available.	4. When the supply in the stream is decreased, the required working head cannot be obtained.
5. The approaches to the excluder are not easily secured.	5. Good approach conditions are easily got.
6. Canal head regulators may be designed to the proper width as usual and they need not be specially widened.	6. The canal regulators will have to carry the discharge meant for escapage.
7. Extraction of silt is managed only once.	7. Multiple extraction is possible, and this can be done by increasing the number of ejectors.

Strengthening of Canal Bank with Silt :—Silt is deposited to strengthen the weak canal banks. This method is adopted where (i) the bank of the canal is of soft and pervious material and is apt to give way by breaching; (ii) the whole channel is in a high bank.

Three systems are generally adopted to strengthen the weak canal banks. They are, (1) Internal silting system. (2) In-and-out system, (3) Long reach system.

Internal Silting System:—This system is simple and economical, and is used for new canals. In this system, from the normal section of the canal, the banks are set back at a little distance. To increase the deposit of silt internally on the berms, inducements are constructed.

INTERNAL SILTING SYSTEM

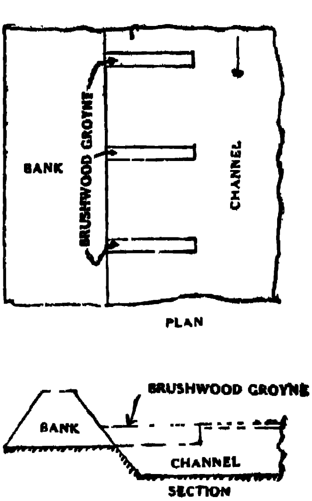


Fig. 260.

This is done by providing brushwood groynes on the water side, having a length equal to the width of berm to be formed by the silt deposit. The top of these brushwood groynes will be at the F. S. L. of the canal and these groynes are kept at suitable distances from one another, along the length of the canal. After a certain time, internal silted berms are formed, the silt being deposited in between every two adjoining groynes, due to the obstruction caused to the flow of water by these groynes. (See Fig. 260).

This method is beneficial in a canal where silt extractor and ejector cannot be economically constructed.

In-and-out System:—In this system, outside the original canal bank (sometimes called primary bank), parallel additional banks (secondary banks) are constructed. Cross banks are put in at intervals varying from 500 to 1000 ft. to connect the primary and secondary banks and thus to form compartments. These compartments are provided with inlets at the upstream ends from the canal and outlets at the downstream ends. A portion of the canal water is allowed into these compartments, which deposits silt there, and the water

In and Out System

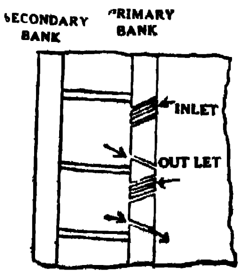


Fig. 261.

free from silt is led through the outlet. When the compartments are fully deposited with the silt, the silt is used for strengthening the land side of the canal banks. (See Fig. 261)

Long Reach System:—As in the in-and-out system, here also, additional parallel banks are provided, with cross banks at intervals varying from 4000 to 5000 ft. Inlets are provided to the compartment at its head and outlets at the tail of the compartment. Also, all the supply of water in the canal is led into the compartment at suitable intervals of time and the main canal blocked at both ends to increase the silt depositing. Spurs can also be adopted, where this system is used. (See Fig. 262)

Long Reach System
Cross Section

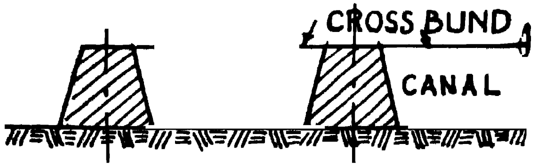


Fig. 262.

Scour—*Introduction*:—Scour is the process of erosion and removal of matter by the friction of running water. The scouring depends upon the velocity of flow of water and on the nature of the material it comes in contact with.

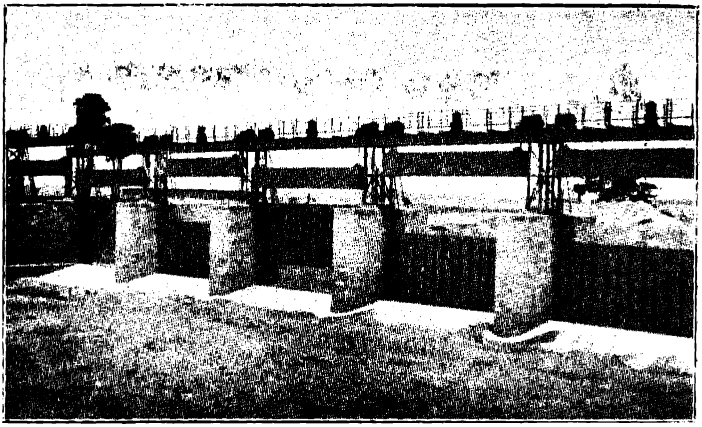
Causes of Scour:—This is due to increase of velocity or sudden change of direction and velocity producing turbulence and cross currents in a certain portion. If the velocity is reduced, silt is deposited in the bed and sides.

Results of Scour:—As a result of scour, the bed of the canal is lowered and there is loss of command. Due to undermining at the toe of the banks, there is a likelihood of the breaching of the bank of the canal. Again if the foundation is undermined, there may be a failure of the structure itself. Due to the marginal scours in a river, it takes a winding course.

All scours must be protected immediately, especially in the case of masonry works, at their foundation. In the case of a canal, the velocity should be made critical. If at places it exceeds the same, necessary rough stone revetment or pitching, or even lining should be done.

Aqueduct

Aqueduct and Escape combined



Mayurakshi Project Deocha Barrage Site

Hirakud Project, Bridge over river Mahanadi.

Comparison of Well and Canal Irrigation:

Well Irrigation	Canal Irrigation
1. Water is to be lifted from the well and hence additional cost.	No lifting of water is necessary, as water flows by gravitation.
2. For lifting water, one has to depend upon mechanism, i. e., there may be breakage of mechanism which means loss of time, and additional expense and also getting skilled labour to do the necessary repairs.	All the troubles mentioned for the well is avoided in a canal.
3. Mostly clear water is got from the wells which is not manurial.	As the water flowing in a canal contains fine silt and clay in suspension, it has good manurial properties.
4. A well lowers the water level, and hence it improves water-logging.	Unless very special care is taken, a canal tends to create or increase water-logging.
5. Well irrigation can be installed wherever necessary. Even isolated places can be easily irrigated.	Canal irrigation can only be done where the canal runs, and only the lands below the canal can be irrigated.
6. Wastage of water can be easily prevented by stopping the process of lifting and hence the loss is small.	When once the head sluice is opened and water let out, all this water should be used, as otherwise there will be loss.
7. The supply to the lands can be made practically constant.	The supply depends upon the flow in the main river and cannot be guaranteed.
8. The assessment is generally fixed by the volume of water used; and so this system is more efficient.	The assessment is usually levied on the basis of the extent of irrigation and hence loss of water and money to Government.
9. Even in times of droughts, well irrigation can be managed, because underground water is always available for use.	When rains fail, there will be no water in the river and canal irrigation is affected.

Comparison of the Cost of Well Irrigation and Canal Irrigation:

Well: Cost of well Rs. 800/– (assumed)

(1) Labour—bullocks and men = Rs. 90 for the season. Four months crop.

(2) Interest on cost of well for four months at 6 per cent
$$= \frac{800 \times 6}{100 \times 3} = \text{Rs. } 16.$$

(3) Cost of maintenance of the field channel = Rs. 12.

Total Rs. 118.

Assume four acres are irrigated by a mote with two bullocks and one man.

Cost for one acre = 118/4 = Rs. 30.

(*Note*: This is in prewar days. It is more now).

Canal: Take Neera canal in Bombay.

Cost of project = Rs. 60 lakhs.

Interest at 4 per cent = Rs. 2·40 lakhs.

Maintenance charges roughly Rs. 60,000.

Total cost = Rs. 3 lakhs.

Discharge in the canal = 140 cusecs, and at a duty 150.

Number of acres irrigated = 21,000.

Note: Cost of Distribution is omitted in both cases.

Rate per acre = 3 lakhs/21,000 = 14·28 Rs.

Hence it is seen that well irrigation is twice as costly as canal irrigation.

The point to be remembered here is that, in the case of large tanks and canals, Government is debited with the whole expenditure on the construction, the interest on the capital and the maintenance of the work: and the revenue derived from the capital frequently does not cover even the maintenance charges: while in the case of well irrigation, the expenditure is mostly incurred by the cultivator in his own interest.

It is to the cultivator's advantage to study the strictest economy in his method of irrigation and it is he who will derive the full benefit. It must also be noted that the above figure indicates that the cost of irrigation by well is much more than that by the

canal system. It is not really so in practice, as irrigation from wells is done by the cultivator only, when he and his bullocks have no other work on hand, and except the initial cost of construction and the subsequent cost of maintenance, the labour charges mean nothing, while in the case of canal irrigation, the cultivator has to pay a fixed water rate in addition to the land revenue.

The cost of a well varies with

(1) the nature of the soil, through which it is excavated,

(2) the depth at which water is found,

(3) the nature and extent of protection required to make it permanently stable and

(4) the material and the labour available for its construction.

Duty of well water: In a canal, the duty is the number of acres per cusec. But in wells, as the discharge is not constant, but variable, it cannot be determined in the same way. This has to be ascertained from a series of experiments. For a *mote*, with lift of 25 to 30 ft. 2·75 to 3·9 acres and for a *lat* with lift of 15 ft. it is one acre.

Loss: In well irrigation, there is a large amount of loss in the water courses, from absorption in the beds and sides of the canal, unless they are lined with some impermeable substance.

In the worst case, it is estimated at 2 c. ft. per ft. run of the watercourse, during a ten hour working day. Though well water is more expensive than canal water, it is more beneficial to crops in some respects.

Questions

SILT

1. What are the different methods of strengthening canal banks by silt from canal water? State their merits and demerits.

Describe In-and-Out method of strengthening canal banks.

(Bombay Univ. B. E. April 1950)

2. Explain with sketches silt ejectors.

(Bombay Univ. B. E. April 50; Poona Univ. B. E. Oct. 52)

3. Discuss the following problems suggesting suitable remedies :

(a) Silting up of reservoirs.

(b) Silting of a canal.

(Bombay Univ. B. E. April 49; Gujarat Univ. B. E. April 53 and Nov. 53)

4. Describe the different methods and devices adopted for preventing silting of canals.

(Bombay Univ. B. E. Oct. 49; Poona Univ. B. E. Civil, Nov. 53)

5. Can the silt deposited in a canal be used to any advantage? Give an example of its use, if any.

(Poona Univ. B. E. Nov. 53)

6. Write short notes on (a) Silt trap and silt ejector and (b) In-and-Out system of strengthening canal banks.

(Poona Univ. B. E., April 54)

7. Distinguish clearly between Silt excluder and Silt ejector.

(Gujarat Univ. 54)

CHANNELS

1. (a) What is "Balancing depth" of cuttings?

Calculate and sketch the typical sections of a distributary of about 300 cusecs capacity with standard berms, when the canal is

(1) Wholly in bank, (2) Wholly in cutting and (3) Half in cutting. Give reasons for the dimensions you adopt.

(b) Why is it that channels are not lined in the Mysore State?

For what situations do you prefer lining of channels? What are the different types of lining in vogue?

(Mysore Univ., Sept. 55)

2. What are 'transit losses' and what are their usual values for the different parts of an irrigation scheme? Under what conditions will the lining of canal be justified?

(Poona Univ., April 55)

3. Define silt. How is the bed silt prevented from entering a main canal taking off from a river in alluvial area ?

(Gujarat Univ., April 55)

4. Draw a neat sketch showing in plan and longitudinal section the Cross Regulator on main canal and mark thereon approximate dimensions of the principal components, assuming your own data. **(Poona Univ. April 54 B. E.)**

5. Explain fully the function of : Canal Escapes.

(Poona Univ. April 54 B. E.)

6. Why and where are Canal Falls required to be provided ? Describe briefly the common varieties of falls in use and Notch Fall in detail.

(Poona Univ. April 1954, B. E.)

7. Write a short note on Canal Falls.

(Mysore Univ. April 1954)

8. Draw sketches to illustrate the type sections for an irrigation canal carrying 200 cusecs (1) wholly in excavation (2) partially in excavation and (3) wholly in embankment.

Why are irrigation canals run in balancing depth of cutting for most of their length ? Why are exceptions made from this rule ? **(Mysore Univ. April 1954)**

9. Explain the difference between . Contour canal and Watershed canal.

(Mysore Univ. April 1954)

10. Draw a typical layout of a canal system from a reservoir and explain the considerations that govern the alignment of the several portions of this distribution system.

Explain briefly Kennedy's silt theory.

(Mysore Univ. April 1954)

11. Write short Notes on Single Bank Canal

(Bombay 1950—Poona 1953)

12. Describe with the help of a sketch the use of Notch Fall.

(Poona Univ. November 1953, B. E.)

13. What do you understand by " Balancing depth of cutting ", with regard to canal design ? What maximum radii do you consider suitable for canals in an alluvial country and why ?

How does the proportioning of bed width to full supply depth affect the design of a canal ?

(Gujarat Univ. April 1953, B. E.)

14. Draw neatly the plan, elevation and section of a Rapid.

(Gujarat Univ. November 1953, B. E.)

15. Distinguish clearly between—Tail reservoir and Tail escape.

(Gujarat Univ. November 1953, B. E.)

16. Explain clearly the difference between—Ridge and Contour canal.

(Mysore Univ. March 1951)

17. Describe briefly Lacey's Theory of Transportation of silt in channels.

(Bombay 1950)

18. State the points you would take into consideration in fixing the alignment of a canal,

Give a type cross section of a canal in high bank.

(Bombay Univ. April, 1949 B. E.)

19. Discuss the following problem, suggesting suitable remedies : Growth of weeds in canals.

(Bombay Univ. April 1949, B. E.)

20. Where do Canal Drops become necessary ? Why is the trapezoidal notch type the best ?

(Mysore Univ. April, 1949)

21. Draw the elevation and section showing a typical pipe outlet on a canal system. How is the size fixed and what is the operation head taken for the design ? Give calculations for determining the diameter of a pipe for the outlet.

(Mysore Univ. March, 1948)

22. (a) Explain the circumstances that lead to the introduction of drops in canals

(b) Sketch your proposal for a six feet fall for a canal of bed width 20 feet and full supply depth 5 feet, combined with a village road bridge of 10 feet width.

All bank connections must be shown.

(Mysore Univ. B. E. Civil. September 1954)

23. What are the main points to be kept in view in aligning a canal for irrigation purposes ? Mention the important data to be collected for properly designing the channel.

Design a canal in fairly hard soil to irrigate 3,000 acres of which three-fifths is sugarcane and the rest is paddy. The bed fall of the canal is 2 feet per mile. Duty for sugar and paddy are 50 and 30 respectively, permissible velocity 3 feet/sec. Value of C in Chezy's Formula = 80, Bed width $= 6d$ (where d = F. S. D.).

(Mysore Univ. B. E. Civil. April 1955)

24. (a) What is " Balancing Depth " of cuttings ?

Calculate and sketch the typical sections of a distributary of about 300 cusecs capacity with standard berms, when the canal is

(1) wholly in bank, (2) wholly in cutting and (3) half in cutting. Give reasons for the dimensions you adopt.

(b) Why is it that the channels are not lined in the Mysore State ?

For what situations do you prefer lining of channels ? What are the different types of lining in vogue ?

(Mysore Univ. B. E. Civil. September 1955)

25 Where do 'canal drops' become necessary ?

Why is the 'Trapezoidal Notch' type the best ?

Give a dimensioned cross section and plan of a notched drop for a fall of 8 feet, the F. S. depth of the canal in the upper and lower reaches being 4 feet. Bed width of the canal = 18′ − 0″. Reasonable assumptions may be made where necessary.

(Mysore Univ. B. E. Civil. 1956)

26 Give a section showing a typical 'Pipe Outlet' on a canal system. How is the size fixed and what is the operating head taken for the design ? Give calculations showing how the diameter of a pipe for the pipe outlet is determined.

Design an outlet from the following data :-

Full Supply Discharge	... 5·00 cusecs
Length of the pipe	... 36 feet
'f' for the pipe	... 0·01
Working Head	... 1 feet

Data not given may be assumed.

(Mysore Univ. B. E. Civil. September 1956)

27 Explain clearly and comment briefly on : Well Irrigation cannot wholly be replaced by Canal Irrigation.

(Poona Univ. B. E. Civil. April 1957)

28 Bring out the difference between

(i) Ridge channel and Contour channel.

(ii) Silt Ejector and Silt Excluder.

(Poona Univ. B. E. Civil. April 1957)

29 Assuming 4 ft. as the F. S. L. depth of flow, determine the bed width and the slope for a Knnedy canal to carry 500 cusecs. The value of C in the Chezy formula v = mi may be taken as 75.

(Mysore Univ. B. E. Civil. April 1957)

30 (a) What are the different methods of aligning canals and under what conditions is each one suitable ?

(b) What main points should be kept in view in laying out the alignments for an irrigation distribution system ?

(Gujarat Univ. April 1957)

31 Explain clearly the necessity of providing falls on canals.

(Gujarat Univ. April 1957)

32 (a) State briefly the Kennedy and Lacey theories and show how the latter is an improvement over the former.

(b) Deduce the relationship $P = 2{\cdot}67\sqrt{Q}$ from Lacey's basic equations, $V_0 = 1{\cdot}1547\sqrt{fR}$ and $Af^2 = 4\,V_0^5$

(Gujarat Univ. April 1957)

CHAPTER XVI

CROSS DRAINAGE WORKS

1. Introduction:—The alignment of all canals and distributary channels is selected in such a way that they run along the ridge and no drainage would thus be intercepted by them. However, sometimes they have to cross streams, when the country is irregular and uneven. Works necessary to dispose of these drainages, are called *Cross Drainage Works.*

The drainage, which any channel is to cross, is disposed of in one of the three following ways:—

(1) The Irrigation canal passes over the drainage. Examples, Aqueduct or Syphon Aqueduct.

(2) The drainage passes over the Irrigation canal. Examples, Superpassage or Syphon.

(3) The drainage enters the canal and mixes with it; Examples, Level crossing–Inlet and outlet.

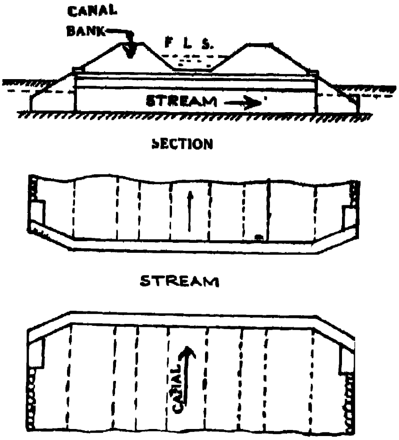
2. Aqueducts: Classification:—An aqueduct literally means a channel for conveying water. It may be either below the ground or above it. But technically, the term is confined to mean a structure spanning a river or other waterway and carrying a canal over it. When a canal is carried over a drainage, without having to drop the bed level of the lower waterway, the work is termed an aqueduct.

Aqueducts in general use are of three different types, according to the topographic and other conditions:—

(1) When the drainage area is small, a tunnel is constructed to drain this water across the canal and the canal banks are continued on the tunnel portion also. The length of the tunnel should be made sufficient to take the two complete earthen banks as per sanctioned section (See Fig. 263).

(2) Another type in use is, in the portion of the crossing of the drainage, masonry walls are also used together with earthwork towards the water side. The object of the masonry wall is to act only as a retaining wall. This wall is constructed flush with the

AQUEDUCT



PLAN AT BOTTOM

Fig. 263.

end of the abutment of the culvert proper. This type is used when the drainage area is moderately large, and the canal section is

AQUEDUCT

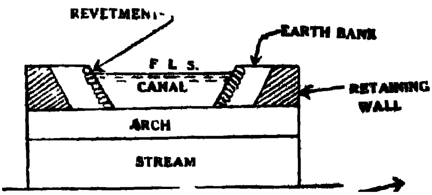


Fig. 264.

SECTION

also fairly large. Towards the front or the water side of the interior earthwork of the canal embankment, pitching is done to the side of the canal (See Fig. 264).

(3) For very big catchment areas and large sized canals complete masonry aqueducts as shown in Fig. 265 are constructed. Here the earthen banks on either side of the main drainage are stopped at the flanks and masonry walls are constructed on the

main aqueduct and continued well into the banks of the canal to have good bond and prevent leakage.

AQUEDUCT, HALL PLAN

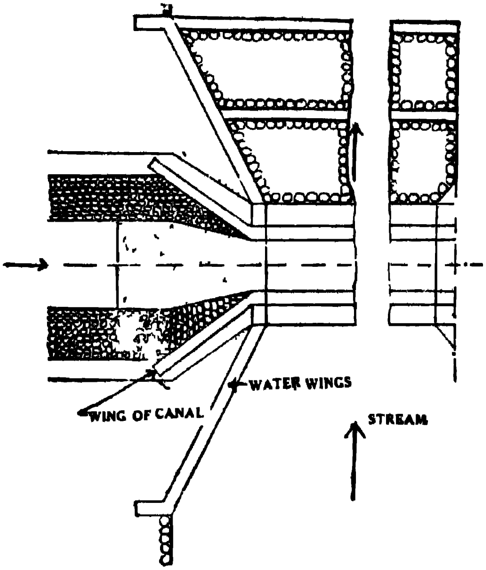


Fig 265.

3. Selection of the site:—The alignment for the river crossing should be carefully selected, so as to give the most economical length of embankment, compatible with straight and permanent reach of the river at right angles to the direction of the canal line. Where there is a long embankment over the valley, special care should be taken to design the aqueduct training works so as to avoid any chance of fluvial action near the embankment.

DESIGN

4. Headway required for the drainage of an aqueduct. The bed level of the channel at the place is first fixed up and the bed of the stream is also known, hence the height available is got from the difference of the two.

The length of the aqueduct should not be narrowed. It should be nearly equal to the width of the original stream.

The discharge in the river is known and the velocity is assumed and the area is thus calculated. The length of the aqueduct and the height available are already fixed as above, and hence allowing for the free board, the flooring and other constructive details and the waterway for the river are fixed.

To determine whether an aqueduct or a syphon aqueduct is to be constructed at a site, the following points will have to be considered :—

(1) If the stream is very large, a syphon aqueduct will be vary costly and if by chance, the river gets a lot of silt, there is the risk of the silt choking the vents and consequent danger to the structure itself.

(2) If the drainage is small, a syphon aqueduct is preferred.

(3) The nature of the foundation and the soil available for banking and also the discharge of any spring likely to be met with, should be considered.

In the case of an aqueduct, the bed of the canal must be at a sufficient height above the bed of the drainage, to allow of sufficient waterway being given in the culvert, to carry the flood discharge of the drainage over which the canal is carried. If the river has cut out a bed deep below the level of the ground through which it takes its course, and its waters in floods are confined within these margins with some free board to spare, there will generally be little difficulty, as regards the sufficiency of headway, and the point of crossing of the irrigation canal may then be selected from other considerations than those of headway, such as, the most convenient and economical alignment of canal or the foundation, the nature of soil available for making banks, the discharge of springs likely to be met with in the foundation, and the uplift pressures to be provided for.

5. The provision of sufficient pitching upstream of bridges and aqueduct is sometimes overlooked. This should be specially attended to, as the swirl of water above a constricted (contracted) waterway has a greater erosive power, than the more noticeable action below it.

The canal water-wings of the aqueduct should be designed long enough to make a thoroughly strong junction with the embankment. The canal-wing of the aqueduct must be made

watertight. A layer of puddle clay covered with fine concrete will be found effective against leakages. The bed should be pitched from erosion by bars of hard material.

Unequal level of foundation is not advisable in aqueduct design, as this would lead to unequal settlement.

Thickness of Abutment :—The width at springing

$$= 2 + \frac{2}{10} \text{ Radius of arch} + \frac{1}{10} \text{ Rise of arch.}$$

Bligh recommends to add to this, one-fifth the depth of water in the canal.

$$\text{Batter of abutment} = \frac{1}{25} \times \frac{\text{Span of arch}}{\text{Rise of arch}}$$

Bank connections consist of double sets of wings ;

(1) Drainage-wings, and (2) Canal-wings.

Drainage wings are intended for retaining and protecting the earth slopes and forming guide walls for the water entering and leaving the work, and also to some extent, acting as stop walls to lengthen the path of percolation of water from the canal above.

The velocity allowed in an aqueduct may be 5 ft. per second or twice that in the canal, whichever is less. This velocity is obtained either by

(1) Increasing the bed fall in the aqueduct section, or

(2) By giving a fall at the beginning of the aqueduct (and hence, here it is protected by pitching).

Contraction of section means, the alteration in velocity which produces extra eddies and disturbance of flow, and therefore not recommended, except when absolutely necessary. Here it is protected from erosion by pitching, etc., generally 25 ft. on either end. The usual heading up at entry is 0·2 to 0·25 ft.

Construction :—(a) All interior faces of walls should be battered to ensure proper junction with earthwork. Usual batters are 1 in. or $1\frac{1}{2}$ in. to 2 in. per ft. of vertical height. (b) Staunching walls should be provided to prevent leakage. At the junction of the earthwork and wing wall, the masonry works should be built in the centre line of the canal bank. These should project from 4′ to 6 ft. into the bank and should be carried at least 1 ft. above the canal F. S. L. and should have a top width of 2 ft. and a batter

of 1/8. (c) Piers, (High piers) should be given a small batter of 2 in. per ft., to gain greater stability and for better appearance.

On the upstream side, the cut waters should be provided by striking arches of circles with centres at opposite sides of the centre lines of piers.

On the downstream side, the piers should be finished off with "*Ease waters*" which in plan may be semicircular or segmental.

The top thickness is usually taken as S/10 to S/5, where, S is the span of the opening.

In the case of abutment piers, the side batters are 1/6.

(d) *Abutment*, width at top, Trautwine's formula :

$$t = \frac{R}{5} + \frac{r}{10} + 2 \text{ ft.}$$

where R is the radius of arch,

r is the rise of arch,

and t is the width at springing

with a batter of 1/8 to 1/6 on the interior face. The back should be filled with puddle and well rammed as the work goes on.

(e) *Wing wall*: The wing wall is to be designed so as to carry the weight of the canal embankment, well out of the direct rush of the stream, through the cross drainage works. The mean width of the section of the wing wall of moderate height is 3/11 of the total height, and for brick work it is 1/3 the height.

If the height is more than 15 ft. it should be fixed by calculation or stress diagram. The splay for wing wall usually allowed is 1/3.

(f) Thickness of arch: Molesworth gives the formula $t = n\sqrt{r}$, where n varies from 0·4 to 0·5 ; another value for n is given by

$$n = 0{\cdot}4 + 0{\cdot}2\,(d-2)$$

where, d is the depth of water in the aqueduct.

(g) Parapet wall of aqueduct should be strong enough to withstand the water pressure due to the depth of water at F. S. L.

For example. For 7 ft. depth, the top width should be 3½ ft. and bottom width 5 ft. 4½ inches.

(Earth pressure = 1/3 the water pressure generally).

Formula in contracted section of an aqueduct :—

$$D = cl\sqrt{2g}\left[\frac{2}{3}\left\{\left(h_e + \frac{V_1^2}{2g}\right)^{\frac{3}{2}} - \left(\frac{V_1^2}{2g}\right)^{\frac{3}{2}}\right\} + d\left(h_e + \frac{V_1^2}{2g}\right)^{\frac{1}{2}}\right]$$

where D is the discharge,

d is the depth of water in the aqueduct,

h is the heading up above the aqueduct, i. e., head of entry,

l is the mean width of the aqueduct, and

c = 0·95 to 0·9

See Fig. 266, showing plan, section and elevation of an aqueduct.

AQUEDUCT

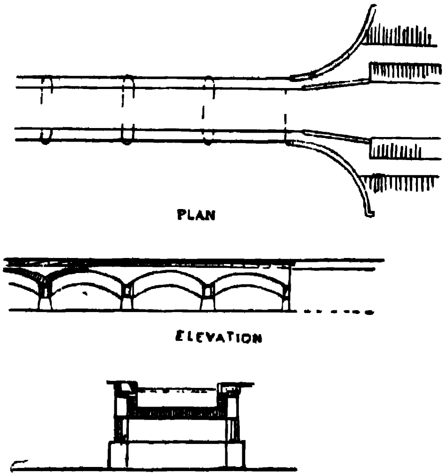


Fig. 266.

(6) **Syphon Aqueduct**:—When the bed level of the drainage is dropped where it passes under the canal and is raised again on the downstream side so that the drainage passes through an inverted syphon, the work is termed *syphon aqueduct*.

A clear headway should be given between the H. F. L. and the bottom of the floor of aqueduct. This should be at least 3 ft.

or half the height of the culvert (whichever is less). This headway is very necessary to prevent blocking up of waterway by silt.* This is an important objection to the adoption of a syphon aqueduct. It is better to use an aqueduct and drop combined than a syphon aqueduct.

Sometimes logs of timber and boulders also choke up the waterway, which is thus silted up.

The velocity through the ventway should be made great in order

(1) to reduce the area, and

(2) to carry away silt, etc.

If the velocity is to be made great, it means creation of more head which may not be generally available. Hence the vent is to be made wider to lessen the depth of excavation.

For a service road, a high level causeway should be provided:

7. Details of Construction:—The following points require necessary attention while constructing a Syphon Aqueduct: See Fig. 267.

* The discharge of a Syphon aqueduct is given by the formula

$$h = \left(1 + f_1 + f_2 \cdot \frac{L}{R}\right) \frac{V^2}{2g}$$

where h is the difference of water level, up and downstream,

L is the length of the barrel in feet.

R is the Hydraulic mean radius of the barrel in feet.

V is the velocity through the barrel in feet per second,

f_1 is a coefficient which provides for the loss of head on entry; it may be taken as 0·505 for an unshaped mouth of the same sectional area.

f_2 is a coefficient which provides for the loss of head due to friction in the barrel = a (1 − b/R). The values of "a" and "b" are as follows;

	Nature of Surface	a	b
(1)	Smooth iron pipe	0·00497	0·084
(2)	Encrusted iron pipe	0 00996	0·084
(3)	Smooth cement plaster or planed wood	0·00316	0·100
(4)	Ashlar or brickwork	0·00401	0·230
(5)	Rubble masonry or stone pitching.	0·00507	0·830

Generally the velocity of appoach is neglected. The interior of a syphon should be cement plastered very smooth.

(1) The drop at the upstream side may be vertical.

(2) The rise at the dowstream side should always be sloping. This is to facilitate the rolling of the silt and debris from the culvert barrel. This slope should not be steeper than 1 in 4.

SYPHON AQUEDUCT

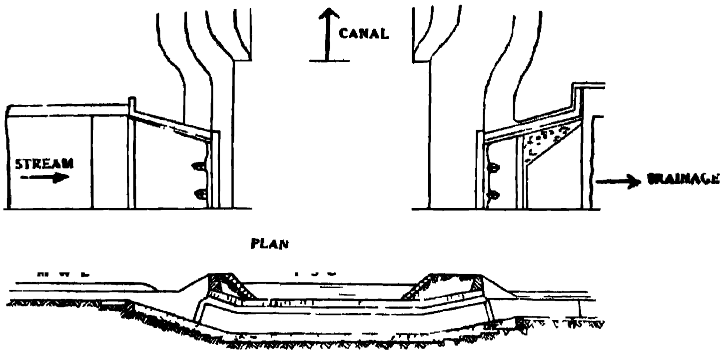


Fig 267.

(3) The width of the cistern between the drop wall and the face of the culvert should be approximately equal to the difference between the M. W. L. and the bed level of the culvert.

(4) Provision for the uplift pressure on the syphon flooring should be made. This is due to

(a) Subsoil water in the drainage bed. The maximum pressure will be when there is no flow in the stream and no water or silt on the syphon floor and the subsoil water is at the bed level of the drainage.

(b) Water in the canal. The pressure is maximum when the canal is full and there is no water in the stream.

To reduce the uplift, the following methods are suggested :—

(i) Provision of puddle apron under the canal bed, both upstream and downstream of the aqueduct. This is to minimise percolation.

(ii) To reduce the uplift due to the subsoil water, inverted arches are constructed in between the piers of the culvert, so that the uplift load is carried by the arch action.

8. Superpassages:—If the drainage is passed over the canal, i. e., the canal goes below the drainage in its normal course, without dropping its bed level, the work is called a Superpassage. A Superpassage is a bridge with parapets high enough to take flood water. Vide Fig. 268.

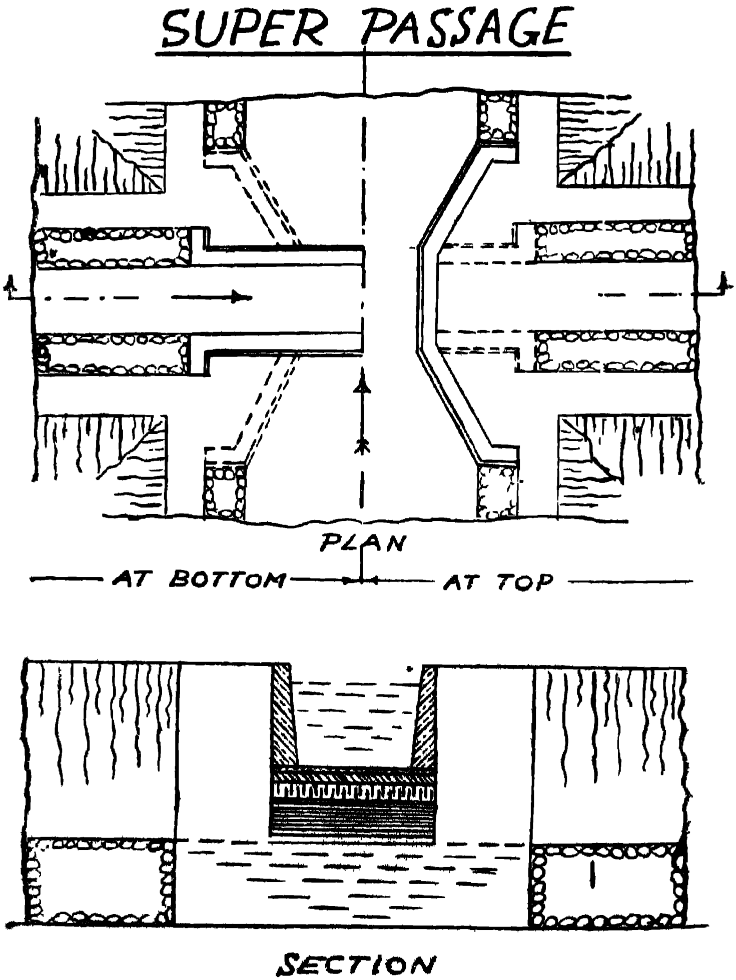


Fig. 268.

Details of construction:—While constructing a Superpassage the following conditions should be attended to:—

(1) Before the design is made, a complete block level of the whole area should be prepared and with all training works and groynes and spurs, if necessary, to maintain the direction of the stream, for the proper approach to the superpassage.

(2) Suitable training works are essential and should be properly designed.

(3) The foundation should, wherever possible, be of mass-concrete, and when this is not practicable, it should be founded on wells.

(4) Usually the drainage will be dry for a long period, and the canal will be full, when there will be considerable bursting pressure on the floor of the superpassage, and provision should be made for this with sufficient thickness (of floor).

The construction should be massive, and the material used should be well selected. The wings should be carried well into the canal banks to avoid the flood breaking round them. The point of crossing should be carefully selected, as also the level of the floor If the canal is to be used as a navigation canal, a lock should be provided if there is not sufficient headway at the crossing of the superpassage.

(5) If the canal carries much silt, sloping approaches should be adopted and the bed and the sides should be pitched and any curtain walls necessary should also be constructed.

(6) If with the superpassage, a canal-fall is also combined, the upstream slope should be protected with puddle and apron, both for the bed and sides.

Advantages :—(1) It keeps the canal free from any influx of flood water.

(2) It does not require the maintenance of any large establishment.

(3) It allows the canal supply to be kept up uninterrupted during floods.

(4) It is useful also as a bridge of communication.

Disadvantages :—(1) It is generally expensive and difficult in construction. The bed of the work (canal) will be at a low

level (below the drainage) and springs are generally met with, and hence proper foundation should be selected.

(2) Necessity for training works, which always require careful watching.

(3) A large cross section of waterway is to be provided over the canal, and it should be safe to carry extraordinary floods also.

Some authorities feel that the structure of a cross drainage may be called superpassage, when the main drainage passes over the canal, irrespective of the change of levels in the bed of the canal; for example, when it is necessary to lower the bed of the canal to cross under a main stream. This is to be called strictly "Canal syphon", but the authorities call even this, a "super passage".

9. Syphons:—This type is suitable where the cross drainage is large, in comparison to the size of the canal. The advantage in a Syphon is that there is no trouble from accumalation of silt as in a Syphon-aqueduct, also no special precaution need be taken regarding the headway. The canal approach to the drainage should be in cutting rather than in embankment. (See Fig. 269).

To resist the bursting pressure on the syphon arches, the following methods are recommended :—

SYPHON SHOWING UPLIFT.

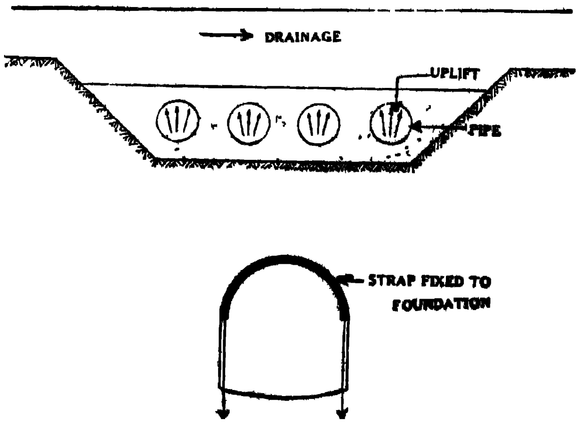


Fig. 269.

(1) The pipe or the vent may be surrounded by thick masonry.

(2) Iron straps may be used to bind the crown to the bottom of the foundation of the structure. These straps should be grouted with cement or given a coat of bitumen to prevent rusting.

(3) The syphon itself or the (pipe) or tube of the syphon may be constructed soley of R. C. C.

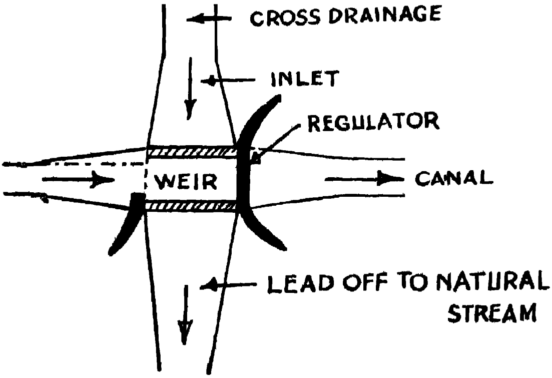
(4) In some cases, thick cast iron tubes may be used and covered on the outer side with a thin shell of concrete, to resist corrosion.

10. Level crossing:—When a canal is taken across and through a drainage at or about the natural level of the drainage bed, it is called a Level crossing. Sometimes, the drainage is received into the canal in the shape of an inlet. This may be an earthen bund (not always safe and may be washed away during every flood).

Drainage water, intercepted by the canal, may be allowed to pass into the canal if

(1) The quantity of water is too small, to warrant the expense necessary for a premanent work to carry it across the canal.

(2) It does not bring any appriciable quantity of water.



SKETCH PLAN OF LEVEL CROSSING

Fig. 270.

(3) It can be passed out of the canal by a special or existing outlet, without the drainage water interfering with the proper regulation of the canal supply.

Level crossing is adopted when the drainage brings small floods, at intervals and where the quantity of silt is small.

The Level crossing consists of an inlet and an outlet and sometimes a cross regulator also. (Fig 270).

Details of construction :—(1) The bed and the slope of the channel should be pitched at the level crossing and to some distance both upstream and downstream.

(2) Below the outlet a defined channel should be provided to lead the water into the natural drainage. The bed and the sides of the channel should be protected. The banks of the main canal on both sides of the outlet should be about 2 ft. above the H- F. L. on the crest of the outlet or weir.

11. Inlet :—These are provided when the drainage area is small, and it does not bring much silt. The inlet should be self-regulating, so as not to require the maintenance of a sparate establishment.

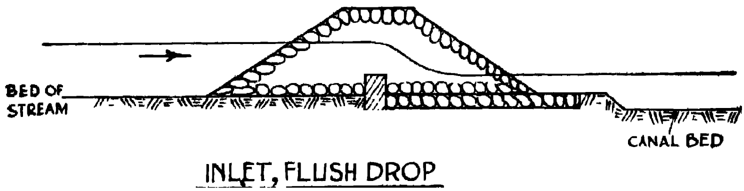


Fig. 271.

The inlet is generally built of the waste weir type and the water is either dropped or stepped into the channel (See Fig. 271).

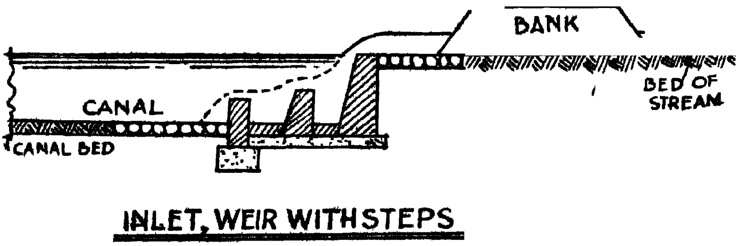


Fig. 272.

The interier of the channel and the sides adjoining should be pitched properly. The inlet is an advantage when the bed of the stream is higher than the bed of the canal or even the full supply level of the canal (See Fig. 272).

When there is an inspection path or a roadway, the inlet should be bridged over.

12. Outlets :—These are escapes with their top level at the full supply level and provided with gates which can be regulated when required. Sometimes these gates are hinged at the bottom and maintained in an upright position by chains, and during floods, they are dropped flat on the sill. On both the sides and the bed of the outlet, proper pitching should be done to protect against any possible scours.

Outlets are built on the downstream side of the channel, either just opposite to the inlet or a little distance lower down. They may be either flush escapes, weirs or sluices with vents.

Outlets are generally classed as

(i) Surface outlets and (ii) Escapes or Surplus sluices.

(i) *Surface Outlets* :—These are mere weirs or flush escapes. These outlets permit of the water to surplus only from above the crest level i. e., the full supply level of the channel. It is sometimes desirable to keep the weir crest somewhat lower than the full supply level. See Fig. 273.

SURFACE OUTLET

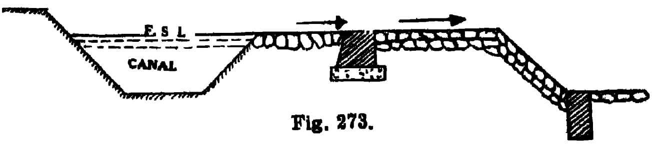


Fig. 273.

(ii) *Surplus sluices* :—In this type, vents are provided in the masonry structure with the sill at the bed level of the channel, so that it will facilitate the complete emptying of the canal and also the scouring of the silt accumulated on the bed. The best location for this type outlet is just at the head of a cross drainage.

The discharging capacity of the outlet or escape is generally taken as half of the main canal. These outlets are specially useful in single bank canals as a safety measure.

13. Cross Regulator. A cross regulator is also constructed in conjunction with the outlet.

The *advantages* of this construction are

(i) It will facilitate the repairs to the channel lower down by closing the regulator.

(ii) The silt above the regulator is easily scoured away.

Hence it is desirable in large canals, to construct outlets with regulators at intervals along the full length of the channel.

14. Objections to the use of Level crossing:—

(1) They bring in silt to the channel.

(2) An operator is necessary to regulate the shutters of the vents of the outlets, unless the shutters are automatic.

15. Advantages of Level crossing.

(1) These are cheaper than other types of cross drainages.

(2) If the drainage is small, and it does not bring in much silt, the extra water can be utilised for irrigation.

(3) Where the cost of a siphon-aqueduct is prohibitive, a level crossing is cheaper.

Questions

(1) What is meant by cross drainage works? Mention briefly (with sketches) the various types of cross drainage works usually required to be constructed on canals.

(A. M. I. E., November 1953)

(2) Describe the general principles governing the design of cross drainage works of canals.

(Bom. Univ. B. E. April, 1950)

(3) What are the different types of cross drainage works used on canals? Under what condition is each type used? Illustrate your answer with sketches.

**(Poona Univ., B. E. Oct., 1952
Mysore Univ. Final B. E. Mar. 51 and Sept. 50)**

(4) Explain with sketches the working of a Level crossing.

(Bom. Univ. B. E. Apr. 1949)

(5) Sketch the plan, elevation and cross section of (a) Superpassage and (b) Masonry aqueducts.

(Bombay Univ. B. E. Apr. 49; Gujarat Univ. 54, and Nov. 53)

(6) Write short notes on Cross Drainage works.

(Mysore Univ. B. E. Final Apr., 54)

(7) Sketch and explain the difference between an Aqueduct and a Syphon.

(Mysore Univ. Final B. E. Apr. 54)

(8) What do you mean by Cross Drainage works in an irrigation channel ? Under what situation would you adopt a syphon ? Draw a typical plan and section of a syphon.

(Mysore Univ. Final B. E. 48)

(9) What are the cross drainage works along a canal ? Describe with sketches the typical situations where an aqueduct, a superpassage and a natural relieving weir are adopted as cross drainage works.

(Mysore Univ. B. E. Final 47)

(10) What are the different cross drainage works that are necessary in the alignment of a canal ? Discuss the conditions where a superpassage and a syphon are used.

(Mysore 45)

(11) A channel, 30 ft. wide and carrying 4 ft. depth of water has to cross a 600 feet wide river.

(a) Suggest a cross drainage work to answer the following conditions.

Bed level of channel	...	R. L. 150·00.
H. F. L. of river	...	R. L. 142·00.
Bed level of the river	...	R. L. 133·00.
Level of river banks at point of crossing	...	R. L. 152·00.

(b) Assume all other data suitably, make necessary calculations and sketch a plan and a section of your proposals, indicating leading dimensions.

(Mysore Univ. B. E. Civil. Apr., 1957)

(12) Sketch in plan, elevation and cross section, a masonry aqueduct, of the type you would adopt while crossing a very wide river. Explain your selection of the type.

(Mysore Univ. B. E. Civil. Sept., 1954)

(13) What are the different types of cross drainage works met with in a canal system? Explain briefly the situations in which each type is best suited.

A canal designed to discharge 40 cusecs crosses a jungle stream, which has practically the same bed level as the channel and 8 sq. miles of catchment basin. Suggest the cross drainage work suitable to the situation giving the reasons. (Assume data not given).

(Mysore Univ. B. E. Civil, Apr. 1955)

(14) Describe with sketches and discuss the conditions when a "Syphon" is used in a canal crossing a drainage. What are the precautions to be taken in such works?

A canal distributary with a discharge of 60 cusecs has to be syphoned under a 30 feet road.

Formation level of road	...	100·00
Canal bed level	...	98·00
Canal Full Supply level	...	101·00
Canal Bed width	...	8·50 ft.

Draw dimentional sketches of the longitudinal section and cross section at the centre of the road. Write a very brief note on the design making any assumptions necessary.

(Mys. Univ. B. E. Civil, Sept. 1955.)

(15) Describe with sketches and discuss the conditions when a 'Syphon' is used in a canal for crossing a drainage. What are the precautions to be taken in such works?

A canal distributary with a discharge of 50 cusecs has to be syphoned under a 30 feet highway.

Formation level of highway	...	2780·00
Canal Bed level ...	...	2778·06
Canal Full supply level	...	2781·00
Canal Bed width ...	...	8·50 ft.

Draw clear dimensioned sketches of the cross section of the road syphon at the centre of the highway. Write a brief note on the design making any assuptions necessary.

(Mys. Univ. B. E. Civil, Sept. 1956)

(16) **Calculate the important dimensions and give a neat dimentioned sketch for a Cross Drainage work to be provided in the case below.**

CANAL		DRAINAGE	
Bed width	60′	Max. discharge	5000 cusecs
F. S. D.	8′	Bed width	150′
Bed Level	505·40	M. F. L.	507·40
Discharge	1300 cusecs	Max. Flood Depth	8′

Hard Muram is available within 5′ from the bed of the stream.

(Poona Univ. B. E. Civil, April 1957)

(17) Draw a clear sketch for a superpassage or an aqueduct and state the precautions necessary for the safety of the structure.

(Gujarat Univ. April 1957)

CHAPTER XVII

WATERLOGGING

(Salt Efflorescence and Malaria)
Salt Efflorescence

1. Introduction :—There are mineral salts in the soil of every agricultural land. Some of these salts help the growth of the crops and others do not allow the plant to grow well. Salts which affect the growth of the crops are also called *alkali salts*. These are brought to the surface of the soil, by the capillary action and evaporotion, and they get deposited to such an extent, as to become visible as patches, and this is harmful to the crops. This process is called salt efflorescence. Kalar, Thar or Shora, Usar and Reh are some of the Indian names given for salt efflorescence on the surface of the soil.

Among these salts, the salts usually met with which are bad, are carbonates, sulphates and chlorides of Sodium. Of these, Sodium Carbonate or '*Black-alkali*' as it is called, is the deadliest. It is called black alkali, because it appears black in solution (which is due to the organic constituents which it contains). But plants can grow even if the soil containg more of Sodium Chloride, for it is less harmful. Sodium chloride and Sodium sulphate are the constituents of *white alkali*.

2. Causes of Efflorescence :—(1) *Over irrigation* :—Due to over irrigation, the water-table in the locality generally rises higher. When this level is within about 4 ft. from the surface, and when the water for irrigation is allowed more than 4 to 5 inches in depth, the water rising in the capilary tubes of the soil, brings salt in solution to the surface, and it is deposited there.

(2) *Impervious substratum* :—If the subsoil is porous, there is no waterlogging, but if otherwise, and also if the existing water level is high, waterlogging is sure to occur and thus the formation of efflorescence.

(3) During summer, water near the surface of the soil is warmer than that at lower depths. Since sodium-sulphate is about

twice as soluble at 35°C than it is at 5°C, the salt from the lower surface moves by diffusion to the surface of the land from the colder water to the warmer water. As the surface evaporation goes on, the salt gets deposited at the surface.

(4) If the hot weather in a locality is for a very long period, due to the heat, the salt rises up to the surface along with the capillary water, and it accumulates thicker when the season is long, than when the season is short.

(5) *Scanty rainfall*: When in a locality there is only poor rainfall, it removes only part of the salts and causes further concentration of the salt. If the rains are heavy, the surface soil would be completely washed away along with the salt formation.

(6) Seepage from irrigation channels fills up the pores in the soil nearby, and gradually raises the subsoil water-table, which naturally causes waterlogging and subsequent efflorescence.

(7) If water is stored for a long period, or it is got by water drained from alkaline lands, it gets saturated with injurious salts. If this water is used for irrigation, the salts present in it get deposited on the surface.

(8) *Improper formation of the land*: If the land is not properly formed, and hollows are left here and there, water stagnates at these places and salt is accumulated.

(9) *Bad underground drainage*: In some localities, high bars of impervious or compact material are found underground across the natural drains. These obstruct the free flow of water underground, and cause waterlogging and efflorescence.

(10) *Basin irrigation*: Where the basin system of irrigation is adopted, a good depth of water is naturally stored on the ground, and as a result the water-table rises, and this is one of the causes of efflorescence. It is advised therefore not to store more than 3 ft.

3. Effects of Salt Efflorescence on Soil: The physical and the chemical nature of the soil is completely changed, if salts are present in abundance in it. The salts get deposited at the surface of the land as explained earlier, and this affects the growth of plants. In some cases, the salts are deposited about 3 to 4 ft. below the surface of the lands, and plants cannot grow in this soil.

and are killed, since the salts have a corrosive effect. This soil which is unproductive and cannot help the growth of plants is called *saline soil*. There will be concentration of salts in such a soil. These soils should be removed early, and if they are left as they are, for some time, usually a reaction takes place between the clay present in the soil and the salt, which affects the fertility of the soil; and by the chemical reactions that have taken place, the extent to which the soil has been affected can be found. This also affects the structure of the soil and changes the moisture relationship.

The soil is affected in two stages. Firstly, the salts occur abundantly which is not very harmful, and secondly the soil, exchanges the base with the salt which makes it impermeable, unproductive and uncultivable, due to the sudamisation (for, the salts are usually sodium salts). This soil is called *alkali soil*, which is more difficult to reclaim than a saline soil. As the alkalinity increases, the reclamation also becomes more and more difficult.

4. Prevention of Salt Efflorescence: The points to be attended to, to prevent salt efflorescence on the surface of lands are.

(1) Evaporation of alkali solutions should be avoided, by using just enough amount of water for irrigation for the growth of plants.

(2) A comprehensive and properly designed surface drainage system on the irrigated land, to drain away the alkali solution, should be provided.

(3) The intensity of irrigation should be low on the land suspected to be affected by salt.

(4) Surface evaporation should be reduced by adopting such cultivation operations which help it.

(5) Alkali water should not be used for irrigation as far as possible. If this cannot be avoided, a large supply of the water available should be given, so that the excess of injurious salts may be drained away.

These preventive measures should be invariably adopted so that agricultural lands are not affected by salts, for as the proverb says, "prevention is better than cure."

5. Reclamation of Salt Affected Lands :—Salt affected lands can be reclaimed by the following methods :—

(1) *Adequate lowering of water-table* :—The water-table should be lowered permanently. First, the sources of water which cause the rise in water-table should be found and they should be cut off by constructing intercept ditches.

Usually in some irrigated areas the rise in water-table will be due to water flowing from higher regions, or from canals or reservoirs, etc. In such cases large open drains should be constructed to lower the water-table. In some cases, the water-table can be lowered by pumping out the excess of water, but this is costly.

A method recently adopted to lower the water level is by Well Point System—see Appendix.

(2) *Washing out the excess of salts* :—Reducing the salt content of the soil, by flooding over the surface of alkali lands is not an effective means. A large amount of water should be applied to alkali lands and made to percolate through the soil, in order to leach away the excess of salts.

Usually the salts that are present in the top 3 to 4 ft. depth of the soil are leached. The water dissolves the salt present, and the salt solution percolates and joins the water table.

(3) *Drainage System* :— Proper drainage system should be provided to drain off the excess of water, thus lowering the water-table and also getting rid of the excess of water immediately after a heavy rain.

The drainage system is divided into

(a) Surface drainage system.

(b) Under-drainage system.

(c) Combined surface and under-drainage system.

The first two are described in detail under Reclamation of Waterlogged areas. The third is a combination of the first and the second, as the name itself indicates.

(4) *Addition of certain salts* :—When certain salts are present in the soil, their effects are neutralised, by using other salts or acids. When the salt-affected soil contains sodium carbonate, chemical treatment is done to remove the alkaline salts before washing the soil. The salt is mixed intimately in the presence of

water with a dose of gypsum ($CaSO_4$) which is added at the rate of about one ton per acre. Reaction takes place between the salts producing calcium carbonate and sodium sulphate. Calcium carbonate is practically insoluble in water and therefore it is not harmful. Sodium sulphate, though harmful, is not so effective as sodium carbonate; moreover it is leached out as before.

(5) *Cultivation of crops which absorb alkali salts* :—Salt-resisting crops are grown on the land, when the amount of salt has been reduced to such a safe limit which the crops can withstand. Coarse rice and Berseen are usually grown, since they have good market value. Berseen is grown in winter and coarse rice in summer. These plants give shade to the land and surface evaporation gets reduced. These crops are grown for one or two seasons, and after all the alkalinity has been removed, that is after the land is reclaimed, ordinary non-salt-resisting crops like wheat, etc. are grown in the land.

(6) *Ploughing the salt crusts* :—The alkali-salt crusts should be tilled deep into the soil. Cultivation operations, such as manure mulching, dry mulching, should be frequently done, which reduce the surface evaporation.

(7) *Management of alkali land* :—Complete reclamation is attained only when the lands are made to produce large crop yields annually. Applying manure liberally, ploughing under cover of crops, avoiding cultivation operation when soil is wet, avoiding intense irrigation but still applying enough water to assure adequate penetration into heavy soils, keeping the drains properly and preventing excess surface evaporation and other important precautions should be considered carefully to reclaim salt-affected lands permanently.

WATERLOGGING

6. Introduction :—When the soil of a land is affected or damaged by too much water, then the land is said to be water-logged. Usually the pores of the soil near the roots of the crop become saturated and thus cut off the normal air circulation around it. When waterlogging occurs, the yield of the crop becomes low. Only aquatic plants can grow in such soils. In some cases, water stands above the surface of the land to a certain height and the land is then called " swampy ".

7. Waterlogging can be detected by (a) The temporary rise in the yield of the crop due to the rise of the water-table which just approaches the roots of the crops,

(b) The rise in the S. S. W. L. in open wells on irrigated area, due to the rise of the water-table.

8. Causes of the Infertility of Waterlogged Lands :— (1) *Anaerobic conditions in the soil* :—Crops require abundant nitrogen for their growth in the form of nitrates. The nitrification is carried out by the bacteria which require oxygen for their work. If the land is waterlogged, it reduces the supply of oxygen and the soil becomes poor with the loss of nitrogen. Thus the anaerobic condition in the soil is indirectly felt by the crop.

(2) *Defects in cultivation* :—This is mainly of two kinds. (a) The cultivation should be done in a limited period as the soil is wet. (b) Good tilth in the soil cannot be obtained as moisture is present in the soil. To remedy this, proper drainage should be provided which allows alternate wetting and drying of the soil and producing good tilth.

(3) *Competition between the crop and the natural flora of waterlogged soils* :—A characteristic kind of aquatic plant grows in waterlogged soils very abundantly. This does not allow the crop to grow well and tries to compete with it. To remedy this, the land should be frequently ploughed, to keep away these plants, and this requires a large expenditure.

(4) *Formation of alkalies* :—The sodium salts present in the soil may be toxic in nature, and this may affect the growth of the crop, or the concentration of these salts may lead to the formation of alkaline conditions in the soil, due to the interaction between the salt and soil, thus producing anaerobic conditions.

9. Causes of Waterlogging :—Waterlogging of lands is the result of the rise of water-table which is due to

(a) The inflow of water in the soil by infiltration from rivers.

(b) The constant inflow of seepage of irrigation water from the adjacent upper regions to the subsoil of the land.

(c) The flow of the seepage of water from the bed and sides of the adjacent canals, tanks etc., which are at a greater height.

to the subsoil of the land. If the soils of the canals, etc. are porous, this inflow causes damage to the land to a great extent.

(d) Percolation of water due to the improper drainage on the surface of the lands, which subsequently raises the water-table.

(e) The lateral flow of subsoil water raising the water-table on the upstream side, due to any impervious obstruction.

(f) Soils with clay substratum, which do not allow easy and natural drainage, increase the level of the water-table with consequent waterlogging.

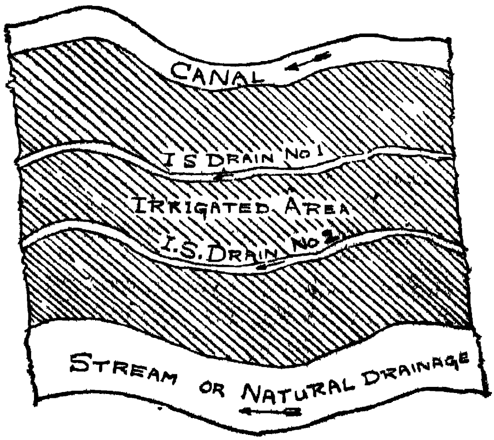
10. Perventive Measures for Waterlogging :—Waterlogging can be prevented by the following methods :—

(1) Reducing the percolation from canals :—This can be done by

(a) *Lining of Canals* :—To prevent percolation from a canal, it should be lined in the leakly reaches. A proper lining generally prevents the trouble.

(b) *Lowering the F. S. L. of the oanals* :—If the soil crust is not cut through, lowering of the F. S. L. of canals is advantageous and reduces the amount of seepage water. If the losses in the

PARALLEL DRAINS TO INTERCEPT SEEPAGE



(Hatched Portion indicates Irrigated Area).
I. S. Drain = Intercepting Seepage Drain
Fig. 274.

irrigation canal are simply due to absorption, lowering should not be done, for it may change the absorption losses into percolation losses.

(c) *Providing intercepting seepage or parallel drains* ;– By providing intercepting seepage drains, waterlogging can be prevented. These drains run parallel to the canal (See Fig. 274) and their principle is sound. They lower the water-table below the lands, up to the seepage water level in the drains. If they are very near to the toe of the bank, they will do their object well, but this procedure increases the percolation loss at the side by steepening the gradients.

(2) Reducing the subsoil flow of the ground-water reservoir :—The measures adopted in this respect are

(a) *Artificial seepage drains* :—To prevent water-logging, artificial seepage drains are provided. They function both as open drains and underground drains, in the form of tile drains. They are very costly, and as such, can be constructed where gravity outfalls into the river are easily available.

(b) *Natural seepage drains* :—The inflow of the subsoil water can be drained through natural seepage drains thus preventing waterlogging. These are provided when the level of the water-table is above that of the lowest water level in the river.

(c) *Pumping the subsoil water* :—The amount of water to be pumped out should be the net subtraction of the ground water, and so it should be able to lower the water-table.

(3) Reducing percolation from irrigation in fields :— This is achieved by

(a) *Restriction of irrigation* :—Irrigation should be restricted by reducing the permissible irrigated area, in waterlogged lands, where the saturated phase of loses below an irrigated field can be established that is, by reducing the intensity of irrigation and by converting perennial canals to Kharif ones.

(b) *Economical use of water* :—This is an important preventive measure in waterlogged areas.

(c) *Lining of water courses* :—This is also a good prevention for waterlogging.

(4) Disposing of the excess of rain water :—The excess of rain water over the crop requirements should be quickly dis-

posed of. For this purpose, surface drains should be provided and this water should be re-used for irrigation.

In some cases, the rain water percolates and becomes underground water, and in such cases, rain is the main factor which contributes to the waterlogging of the land.

N. B. :—The replacement or supplementing of the canal irrigation by the tube-well irrigation is an effective prevention for waterlogging. (Vide Chapter on Tube-wells).

11. General :—*Inflow* is the flow of seepage water into the subsoil of the land. The factors which contribute to the inflow are

(1) Percolation from canals,
(2) Subsoil flow in the ground water reservoir,
(3) Percolation from irrigation fields,
(4) Percolation from rainfall.

Outflow is the flow from the soil which lowers the watertable. This is effected by

(1) Infiltration into the rivers,
(2) Subsoil flow to the lower regions,
(3) Soil evaporotion from the water-table surface,
(4) Transpiration by the plants,
(5) Seepage drains.

The inflow and outflow should balance each other, if the water-table should be in equilibrium. The outflow will automatically get reduced if the inflow is reduced. To reduce the outflow, the head above the river should become less, thus lowering the water-table. So, to remedy waterlogging, the amount of inflow should be reduced.

12. Reclamation of Waterlogged Areas :—The waterlogged area can be cured and made productive by providing artificial surface drains, and sub-surface drains, thus lowering the water-table below the root zone of the plants. This process is called reclamation of waterlogged lands. The water-table should be lowered at least 5 ft. below the surface of the land, for the crops to grow well. To remove the excess of water from the waterlogged lands, surface open drains of trapezoidal cross section are constructed. The sub-surface drainage consists of

(a) Pipe drains (Fig. 275a), and

(b) Deep open drains (Fig. 275b).

LONGITUDINAL SECTION CROSS SECTION

PIPE DRAIN

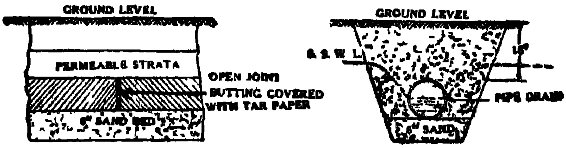


Fig. 275 (a).

OPEN DRAIN

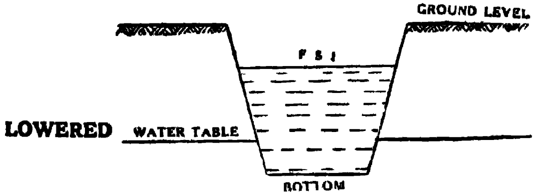


Fig. 275 (b).

(a) The pipe drains should be placed below the surface of the land in the permeable saturated stratum. The pipes are usually of vitrified clay, and are placed on a 6 in. sand bed, with open joints butting against each other. To prevent the sand and the earth from entering the pipe, through the open joint, a tarred paper is provided to cover the joints. These pipes are placed in the trenches with a proper slope and the trenches are filled with the excavated earth and sand so that the cultivation operations can be conducted over them. The maximum depth to which the pipe drains can be lowered below the ground is about 18 in. exceeding which, there is the possibility of their breaking under the weight of the earth over it.

The pipe drains should be constructed at a level, below the water table, about 2 ft. below the level to which the water level is to be lowered.

(b) Deep open drains, which are cheaper, are constructed where a drain of bigger diameter than the maximum extent to

which the pipe drain can be built is required. The open drains should be deep enough, so that the water-table can be traversed. Its bottom should be below the level to which the water table has to be lowerd, so that its full supply level will be below the ground level, and thus waterlogging may be prevented.

The sub-surface drains are distinguished according to their work as

(1) *Relief Drains* : These are provided to remove the superfluous water in the saturated soil, which flows into them. These may be pipe drains or open drains. (Open drain is used where the water to be drained is much).

(2) *Carrier Drains* :—These are bigger in size and the water from the relief drains flows into them. They may be open drains or pipe drains. They are laid on impermeable stratum and sometimes they do the work of relief drains also.

(3) *Intercepting Drains* .—Refer to Preventive Measures for Waterlogging.

PLAN OF DRAINAGE

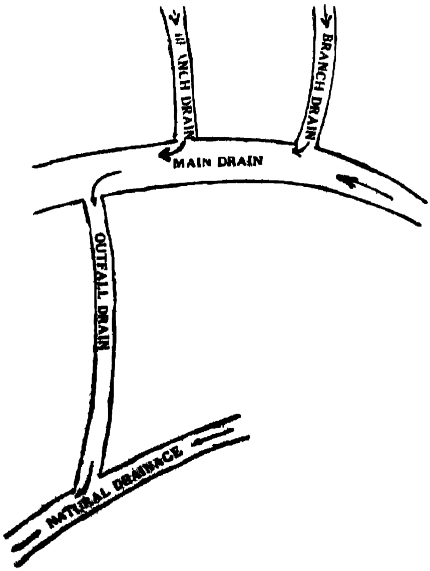


Fig. 276.

Branch drains, main drains, and outfall drains, constitute the whole under-drainage system. Branch drains are usually relief drains, small in size and they are provided at the extreme top of the land. The main drains are usually carrier drains, bigger in size than the branch drains and the water from the branch drains flow into them. Outfall drains are open drains to which the water from the main drain flows. The water from this is discharged into the natural drainage (See Fig. 276). The depth to which the water table is to be lowered and the nature of the soil to be drained are the factors on which the time taken for the reclamation of a land depends.

13. Project for draining the waterlogged area :—Waterlogged areas cannot be left as they are, because they lead to the development of Malaria, due to the stagnation of water, and if this area is drained and improved, it will not only prevent the dirty disease, but also produce some food crop. Hence the necessity for the project.

To drain the area, two things must be known. The line of flow of the ground water and the surface, upper and lower—or the thickness of the pervious stratum (murum). When these two things are known, drainage can be provided at the top surface of the murum and the waterlogged area allowed to drain into the water table through the porous stratum.

The line of flow of the subsoil water is called the *Hydro isobath* and the configuration of the layer of murum at the place is called *Murum isobath.*

To know the level of the underground water and the level of the surface of murum, bore holes have to be drilled. For this purpose the necessary tools, such as, boring auger, shovels, etc. will have to be got.* When the water first rushes through the bore hole, this level should be specially noted as it is required for purposes of investigation. This level is called the First level of strong Flow.

First a rough survey of the whole area is to be made and some stray bore holes drilled and the required information got and a preliminary proposal submitted and after it is scrutinised and approved or modified by the responsible officer, detailed proposals

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* For detailed study, refer to author's book on "Advanced Construction."

should be submitted along with the required drawings and estimate as noted below :

(1) A general index plan.

(2) A plan showing the area affected and to be reclaimed.

(3) A big scale plan, showing the subsoil conditions in the area (water-table, hydro isobaths, murum isobaths, substrata, etc.).

(4) A longitudinal section of the natural underground water-table, showing in it, the drains proposed.

(5) Detailed drawings of any important masonry works.

14. Construction of drains :—Before the actual construction of the drains is done, the line must be correctly aligned. This should be done, taking into consideration the line of flow of the ground water and the pervious strata. To lead the water to the outfall, the drainage should be given a proper alignment and gradient, and wherever there is a change either in the alignment or in the gradient, a manhole should be erected.

Regarding the design of the drains, sufficient information should be gathered regarding the actual seepage at the line and any seepage coming from the adjacent lands and also any rainfall, and a provision of an addition of 50% of the total amount as got above, should be added to the total quantity to be discharged through the pipes, or the open drains according to requirements. If the drains are open, provision should also be made for necessary weed clearance.

MALARIA

15. Malaria and the Engineer :—It is said that man made Malaria by blocking the drainage and creating swamps, for the breeding of mosquitoes to develop Malaria. As such, when any big irrigation work is taken up special provision should be made for meeting this requirement. Malaria is not a disease of today. It is said that the fall of the Roman Empire was due to the low vitality of its soldiers who were badly affected by Malaria in Asia and Egypt. (Mosquito nets were in existence even in the time of Herodotus and Cæsar.) Lesseps, the French Engineer, who after Suez canal projected the Panama canal first and failed, called the Panama canal zone the white man's grave. This was due to the Malaria at the place.

16. Causes of Malaria:—Malaria is caused by the injection of Malarial poison into the human body. For this purpose there should be (a) a Store, (b) a Carrier and (c) a Living (human) body. The poison is stored in the Malarial patient. A mosquito carries it from a patient to a man free from it and injects the same into his system. For the mosquito to live and breathe, a living place is necessary and this is mostly water.

The mosquito which injects Malaria is the Anopheline. The insect requires (a) Breeding place, (b) Food preference, and (c) Range of flight.

Breeding place:—The insect thrives not only in stagnant water, but also in running water. Different species live in differeent kinds.

(i) Some mosquitoes breed in fresh water and they prefer pooled intermitent streams, borrow pits near the roads and grassy ponds.

(ii) Some live in salt water, especially in salt marshes.

(iii) Some live in slightly alkaline water.

(iv) Some live in acid water.

(v) And some other species have seasonal migratory flights.

Food:—The male mosquito lives only on vegetable food, fruits and flowers, but the female mosquito requires blood and mostly human blood.

Flight:—Mosquitoes have a limited range of flight depending upon the climate. This should be found by experiments. The mosquito is usually said to fly about two furlongs. In practice, while restricting the boundary for irrigation for rural areas, two furlongs zone is adopted and for towns four furlongs.

All irrigation works do not generally bring in Malaria. For example, Lift irrigation and Minor tank irrigation. It is only in major irrigation works, where proper attention is not paid to drainage, that Malaria flourishes. The Irwin canal, i. e, Visveswaraya canal from Krishnaraja Sagara, Mysore, was opened in 1932 and water was allowed to flow freely and it was allowed to stagnate here and there, and thus Malaria began its swing in 1933.

The causes for the development of Malaria in an irrigated area are the following:—

(a) Leakage from sluice gates.

(b) Seepage from canals.

(c) Ill-maintained channels.
(d) Borrow pits.
(e) Scour of the bed and the sides of channels.
(f) Field channels not properly planned and controlled.
(g) Fallow rice fields.
(h) Bad drainage and the natural drainage when obstructed.
(i) Over-irrigation.
(j) Water is stagnant in a paddy field and it flows slowly and the water is renewed only once in two or three days. This stagnation combined with the shade of the plants encourages the growth of mosquitoes.

(k) River embankments and dams raise the flood water level and the bed of river by silting up, and the water percolating through the dam creates marshes. These marshes are further worsened by drainage which cannot reach the main river due to the flood embankment and spread Malaria.

17. Places of Harbour for Mosquitoes in Irrigated Areas.—The place where the mosquitoes usually harbour are .—

(a) Borrow pits.
(b) Tanks.
(c) Wet fallow lands.
(d) Valleys with intermitent flow and vegetation.
(e) Seepage area.

18. Prevention of Malaria:—Malaria can be prevented by the following methods .—

(a) Use of mosquito curtains.

(b) Spraying the houses with Pyrethrum (This is costly).

(c) Oiling the stagnant water.

(a) There should be a Malaria patient for the mosquito, if not, it cannot inject malaria, since there is no store of poison. So if all the malarial patients are provided with mosquito-proof houses, malaria is prevented to a very good extent.

(b) Pyrethrum with a mixture of Kerosene oil should be sprayed in the houses bi-weekly, after the windows are shut in the evening, which will reduce the breeding of the mosquitoes. It is said that it takes about a fortnight for a parasite to develop, in the body of the mosquito.

(c) Oiling the stagnant water with Paris green and other oils makes the water unfit for the breeding of the mosquitoes.

With a view to checking the breeding of mosquitoes in borrow pits, the Punjab Public Health Department has invented an oil mixture for oiling water collection, which is believed to be useful as a mosquito larvicide. The formula of the oil mixture is; Crude oil one part, Kerosene oil four parts and Castor oil 0·1 part. The quantity of oil necessary to control mosquito breeding varies generally according to the local conditions obtaining, i. e., climate conditions, presence or absence of vegetation, kind of oil or mixture of oils employed, etc. As a general rule, however, half an ounce of oil per square yard or about $5\frac{1}{2}$ ounces per 100 square feet is usually an ample estimate. The exact amount of oil required in any particular case can be determined by actual trial.

19. Remedies of Malaria :—*Provision of proper drainage*—This is effected by the subsoil drainage of the seepage area, lowering the existing tunnel if required so as to allow free flow of water, and in some cases, construction of new tunnels for embankments, and generally by filling up all pits and draining them. If pits cannot be drained they are sprinkled with Larvicides like Oil emulsion and Paris Green at regular intervals. Not only pits but also old discarded tin cans, broken crockery, and conjested drains harbour mosquitoes. These should be prevented.

(2) *Canalisation of valleys* :—For this purpose, the existing nalas or valleys should be properly graded and canalised. Also, they should be flushed periodically by automatic or hand operated flushes.

(3) *Depletion of tanks* :—The tanks in the irrigated area, should be either breached or provided with deep level scouring sluices, to deplete the water when required.

(4) *Sudden lowering of water level* (between high flood level and the full tank level):—Mosquitoes generally harbour at the boundary of the water-spread and hence when the water is suddenly lowered, mosquitoes perish.

(5) *Shifting of villages* :—To prevent the infection of Malaria, villages will have to be shifted away higher up from the waterlogged area.

(6) In Indonesia *fish ponds* are maintained where mosquito-eating fish are bred and passed on with the water to the paddy fields.

(7) *Natural methods*:—Nature has her own methods of control. Upper Egypt is free from Malaria due to the cultivation of (a) Egyptian clover, (b) Wild Melitatus Trigonella, (c) Coumorin, and similar herbs.

(8) *Improving the sanitation of villages*:—Cheap surface drains having a narrow bottom width, to allow sufficient velocity should be built and all manure pits should be well sealed; also provision should be made for good drinking-water wells.

(9) *Improvement of communication*:—Construction of inter-village roads with culverts wherever necessary, should be expedited.

(10) *Improving channels near villages*:—The channels near a village should be either closed or deviated so that they may be aligned below the village and away from it. (They should not be above the village, in which case there will be seepage from the channels,) and wherever possible, the channels must be provided with lining.

20. Comparison between Tanjore and Mysore areas of irrigation:—

Tanjore	Mysore
This is situated in a plain country and in delta region.	This is situated on a plateau with ridges and valleys.
There is only one kind of Anopheline thriving here.	There are two kinds of Anopheline. These are more potent and are Malaria carriers.
There is intense cultivation and no lands are left fallow.	Due to want of labour mostly, a good amount of lands is left fallow.
Lands and the surroundings are kept scrupulously clean.	Lands and surroundings are kept very bad.

21. Malaria and Remedies suggested :—Impounded water assumes an important position in respect of health due to its potential breeding capacity for the deadly *Anopheles Mosquitoes*–the vector for Malaria. These mosquitoes do not usually develop in open water free from vegetation or flotage. This fact is fundamental in the control of impounded water. As such, an absolute minimum of flotage and vegetable growth should be allowed. In any project, the area proposed to be impounded should be completely denuded of all vegetation, which may in course of time become flotage. This may include all types of woody vegetation, tree trunk, and roots and bush of any sort. And all these should be removed well beyond the H. F. L. of the reservoir long before the impounding begins.

The water spread area of the reservoir fluctuates with the level of water in it. This exposes and encourages growth of marginal vegetation, which generally is conducive to larvae development, and so should be kept under proper control.

Even in the bed of the reservoir all possible pools where water may stagnate should be avoided. These pools collect flotage increase the total shore line of the reservoir and are a source of refuge for the mosquito. These pits should be either filled up or drained away before the actual impounding of the reservoir begins.

The reservoir not only provides the required storage, but in addition, it produces other advantages as noted below. The area of the bed of the reservoir is advantageously stocked with game and commercial varieties of fish, and these become assets to the National Economy and also they provide sport to the people.

The impounded water produces an odour due to Hydrogen Sulphide gas, which sometimes hinders visitors coming to the reservoir. In addition to the above disadvantage, Hydrogen Sulphide absorbed in water discolours it, and at the same time, minute quantity of Sulphuric Acid is formed. This is dangerous to fish-growth, and in addition the acid causes skin eruption for those who use the water for bath. Unfortunately this acid is not enough in quantity to kill the mosquito larvae.

The acid, though small in quantity, causes serious damage to the structure where concrete and steel reinforcements are in use, because Sulphuric Acid will corrode and destroy both these materials.

The products got from the bed of the reservoir (removal of trees, jungle, etc.) have commercial values which give a good return to the State. These products may be (i) timber which is marketable or even used for construction work in the project itself and (ii) Charcoal, which may also be marketable or used for the project or by the labour working there.

Even after the project is sanctioned, until the project is completed (it may take a number of years), the bed of the reservoir can be easily converted for the growth of agricultural crops, after the removal of the forest growth there. The agricultural products grown here, may be food-crops which is useful for the men employed on the project.

Even after the completion of the project, the shore area from which the water recedes for some period annually may be used for agriculture. This procedure not only is useful for the control of mosquito but also provides food for the people.

Hence for the proper eradication of malaria from the region of the reservoir, co-operation among the Irrigation Engineers and the Forest and Public Health Organisations is very essential.

Questions

(1) Which proves injurious for agricultural land and when? How does such land become sodamised and how can it be reclaimed? **(Gujarat Univ., April 55)**

(2) Define waterlogging. How can it be detected? Why does an agricultural land become waterlogged? How can water-logged land be made productive? **(Gujarat Univ., April 55)**

(3) Enumerate the reasons which lead to waterlogging in irrigated areas and explain how drainage helps to bring about physical improvement of soil and encourages growth of crops. **(Poona Univ., April 1955)**

(4) What is meant by waterlogging? Explain how and why it occurs and give the methods of reclaiming waterlogged area. What exact purpose does the H. I. B., and M. I. B. serve in this connection? **(Gujarat Univ. 1954)**

(5) Describe the causes of salt efflorescence and their treatment. What precautions should be taken to prevent salt efflorescence in an area where irrigation is to be newly introduced?

(6) Write a short note on Hydro Isobath.

(Dept. of Tech. Ed., 1954)

(7) What do you understand by "waterlogging," in irrigated tracts and what are the factors responsible for infertility of water-logged soils? **(A. M. I. E., May 1953)**

(8) Write a short note on waterlogging in irrigated tracts, its effect on the fertility of the soil and the methods of reclaiming such lands.

(A. M. I. E., Nov. 1953)

(9) What is waterlogging? State the reasons and the main factors that affect the subsoil water-table in the Bombay and Deccan areas.

(Bombay Univ. 1953)

(10) In an irrigated area, how is land damaged by water-logging and salt efflorescence? Describe briefly how such damage is controlled and how such damaged areas are reclaimed.

(Gujarat Univ. April 1957)

(11) What are the causes of waterlogging in the Deccan canal areas?

What do you understand by (i) relief drains. (ii) carrier drains, and (iii) intercepting drains?

(Poona Univ. Oct.. 1952)

(12) Write a short note on waterlogging in irrigated tracts and methods reclaiming such lands.

(A. M. I. E. November 52)

(13) Write a short note on Salt efflorescence.

(Poona Univ. Oct. 1952)

(14) (a) Define waterlogging of a tract.

(b) What are the principal causes of waterlogging in a canal irrigation tract?

(c) What precautions will you observe in constructing a new canal system?

(d) What steps will you take to improve an already waterlogged tract? **(A. M. I. E. Nov. 1951)**

(15) What are the different types of permanent antimalarial engineering works you would suggest in an irrigated tract? Explain in detail their functions with neat sketches.

(Mysore Univ. B. E. Final Sept. 50)

(16) Explain with sketches wherever necessary the Anti-malarial Engineering works.

(Mysore Univ. B. E. Final, April 49 and 47)

(17) The irrigated tract under a big canal has become highly malarial with portions of the land waterlogged. What remedial measures would you recommend to improve the situation effectively and to check the recurrence of such conditions.

(Mysore Univ. B. E. Final 1946)

(18) Why do irrigated soils get damaged? Briefly state if and how they can be reclaimed. Give sketches.

(19) Explain clearly and comment briefly on

(a) Profuse and indiscriminate use of water (irrigation water) causes harm to all concerned.

(b) Certain crops can be raised in a salt-affected soil at times.

(Poona Univ., April 1957)

(20) Write short notes on

Water-logging and causes of salt efflorescence.

(Gujarat Univ. April 1957)

CHAPTER XVIII

EVAPORATION AND ABSORPTION

Loss by evaporation and absorption :—

Effect on the discharge in different cases :—

1. **(A) Reservoir :** Losses in reservoirs are mainly due to (1) *Leakages*, (2) *Evaporation* and (3) *Absorption.*

Leakage is measured by the following methods :—(a) Pipe, (b) Notch, and (c) Cippoletti weir. Evaporation is measured by the Box method. Loss due to absorption is got by deducting the losses due to leakage and evaporation, from the total loss of the reservoir.

Total loss is given by the formula $P + Q - R$,
where P is the old capacity of the reservoir
Q is the inflow into the reservoir
R is the outflow from the reservoir.

The amount of evaporation depends upon (a) Heating effects of the sun or the temperature and (b) Drying effects of the wind, or the rate at which the vapour is removed from the surface by the wind. The loss due to the second is greater than in the first case. A high temperature has a considerable effect and ordinarily, there is more evaporation from shallow waters than from deep ones, because the temperature rises higher in the shallow water. [In the case of a small cistern used for experimental purposes, the loss due to the drying effect of the wind is not felt]. Evaporation appears to proceed even at night times, almost the same as during the day time.

The loss due to evaporation varies,

(a) Directly, as the area of water surface

(b) Inversely, as the depth of water.

If E is the loss due to evaporation, A is the area of water surface and *d* is the depth of water, then $E \propto A/d$.

Loss due to evaporation cannot be prevented. Loss due to absorption and leakage can be prevented by using watertight

material, cement plastering where leakages occur and providing a puddle or concrete core wall (in all earthen dams).

Loss due to absorption alone depends upon,

(a) the nature of the bed of the reservoir.

(b) the pressure due to the depth of storage.

The first condition increases with the porosity of the underlying strata, while the second increases with the pressure due to the depth of water.

In ordinary cases, after the bed of a reservoir becomes waterlogged, the loss from absorption will be generally much less than that from evaporation. Hence deep reservoirs, which present comparatively a small surface, will suffer less total loss than shallow ones.

In the loss by absorption is included the loss due to leakage under the dam. (This is prevented by the construction of a puddle trench).

The total loss due to evaporation and absorption varies with

(a) the period during which these causes act.

(b) the season.

In the monsoon, the air is humid, but there is a strong wind at times, which causes some evaporation. Besides, the rainfall in the basin will make up for the evaporation.

The following table shows the evaporation during the eight months of the year.

Month	October	November	December	January	February	March	April	May	Total
Evaporation in ft.	0·25′	0·19′	0·14′	0·17′	0·14′	0·17′	0·27′	0·38′	4·34′

In cold weather, evaporation will not be great, as then, the temperature will be low, and also there is no high wind.

From the table, it is seen that this gives nearly 4·34′ for the period of eight months and represents the total loss due to evaporation and also absorption. Loss from evaporation rarely

exceeds 0·4 in. a day in the hottest and driest part of India. Loss from absorption varies with the nature of the bed of the reservoir, as said already, and may be taken as not more than half the loss due to evaporation. An example is given below.

Ekruk Tank **(Sholapur)** :—Here the loss due to absorption and evaporation in hot months (April and May) was found to be 0·384 inch a day, while in the cold months, (November to March), it was 0·232 inch a day.

For the preparation of projects in the Ghat area evaporation and absorption loss is taken at 4 ft. on the mean area at F. S. L, and outlet level.

5 ft. ... ... 25 miles from the ghats
and 6 ft. ... ... 50 miles from the ghats.

N. B..—6 ft. is generally assumed in India.

The sill of sluice or outlet should be kept above the lowest bed of the nalla (by a reasonable depth 5 to 10 ft. according to the capacity of the reservoir). This is for any silt coming from the catchment to accumulate. This silt deposit acts like a cementing material and makes the bed safe against percolation and absorption.

2. (B) Canals:—The quantity of water lost in a canal varies with the nature of the soil in which the canal is cut, and with the spring level of the subsoil. The following conclusions have been deduced from experiments.

Absorption:—

(1) Loss by absorption is greater when a canal is in cutting than when it is in a bank.

(2) Other conditions being constant, the loss by absorption varies directly as the wetted perimeter of the channel.

(3) Under ordinary conditions, the loss by absorption between the head of a distributary at any point l miles from the head, can approximately be ascertained by the formula

$$\text{Loss} = Al^x$$

where A is the loss actually ascertained by experiment in the first mile

x is the power to which l is raised. (x varies from 5/6 to 6/7).

l is the distance in miles from the head.

(4) That there is more waste by absorption in village channels, than in all other parts of an agricultural system.

(5) That the loss by absorption is much greater in a new canal than in an old one, as here the porous surface of the absorbent soil becomes coated with silt, which is deposited and is driven into the interspaces in the ground.

The more approved formula for the loss of absorption in the Punjab Irrigation department is

$$P = C\sqrt{d}\ \frac{Wl}{1000000}$$

i. e., $P = 3\sqrt{d}$ per million sq. ft. of water surface

where P is the loss in cusecs by absorption in any reach,
C is a constant usually 3·5,
d is the depth of water supply in ft.,
W is the width of water surface of reach in ft.,
l is the length of reach in ft.

Evaporation:—Examples.

Experiments conducted on the Ganga canal gave the following results for evaporation:—

The canal in question is 50 ft. wide and 5 miles long and gave 1·65 cusecs as loss due to evaporation which is very small, compared to the canal discharge of 600 cusecs.

February	0·14″ a day
March	0·12″ „
April	0·15″ „
May	0·12″ „
June	0·12″ „
October	0·13″ „
December	0·10″ „
Mean	0·13″ per day

Nira Canal (Bombay) :—Length of the canal experimented was 100 miles and discharge 455 cusecs. The average loss amounted to 1 cusec per mile or 100 cusecs in all, which is equal to 22%.

In the Mootha canal, the loss was seen to be 0·8 to 0·9 c. ft./sec./ mile.

3. (C) Distributary :—The condition in a distributary channel is different. The excavation for the channel does not go deep into the soil strata. The channels here are more stable and generally become lined with a fairly impermeable clay silt coating, especially if they are designed of proper capacity and well maintained, and the loss by evaporatian is also very little.

If, however, the distributaries are kept alternately wet and dry for short periods, there is an appreciable loss by soakage and percolation.

Evaporation from distributaries is probably less than from the main canal, as the banks protect the surface of water from the action of wind and also the shade of trees on the banks reduces evaporation.

4. (D) Field channels or Water courses. There is great loss by percolation in water courses, as the channel is generally daily opened and closed. The loss from improperly aligned and badly constructed water courses is very great indeed, and as far as economy of water is concerned, there is more to be gained by proper attention to these channels.

5. (E) Streams. In a stream where the bed is hard and compact, there is an increase in the discharge at a lower section than at a higher one, indicating that the gain by percolation is more than the loss by evaporation. This is not the case in broad shallow rivers, and in sandy beds, but would happen in deep-flowing rivers with hard impermeable beds.

It may therefore prove economical in certain cases, to carry water in the natural bed, in preference to a high level artificial channel. The loss from percolation in large artificial channels passing through sandy or porous soil is very great, until the soil between the surface of the ground and the subsoil water level has been saturated to its full extent. Thereafter the loss will be very little compared with that previously.

6. (F) Projects., It would be sufficient for project calculations to allow 0·10% of the total discharge per mile for all the combined losses due to evaporation and absorption in canals, which have been running for a long time.

Evaporation from water surface is greater than that from land. It is greater in dry and desert regions than in cultivated ones; it is also greater in regions where the rainfall is 40 in. than at a place where the rainfall is 60 in. Evaporation is less in low lands, than in mountains. Evaporation decreases as the humidity of air increases.

Experiments :—Pan 22 in. diameter, 28 in. deep sunk in earth. Evaporation increases with the temperature as shown below :—

Temperature	53	62	73	80	88
Evaporation	6	12	23	31	39

Evaporation also increases with the velocity of the wind.

Wind Velocity	5	10	15	30
Evaporation	2·2	3·8	4·9	6·3

CHAPTER XIX

ADMINISTRATIVE AND FINANCIAL ASPECTS OF IRRIGATION WORKS

1. General :—All the works should be carefully estimated to begin with and it should be seen that the amount of estimate is made as minimum as possible, consistent with the safety of the structure. It should also be seen that any project work is not a loss to the Government, but it should pay something in return.

Irrigation works are classed as major and minor (Original Irrigation Major, and Original Irrigation Minor), according to the magnitude of the project. The amount required for the schemes is alloted by the State Government, and generally a decent grant is also given by the Central Government. The big projects, now taken up under the Five Year Plan, are mostly managed by the Centre.

An estimate of the cost of the project is first got prepared. It is then sent up to the sanctioning authority. (The sanctioning authorities are different for the amounts involved in the project) and after it is got sanctioned, the work is started, either departmentally or given to a contractor after calling for tenders, and then the work is completed accordingly. After the construction of the work, it is important that it should be maintained in good condition. Both the construction work, and later on, the maintenance work, should be done economically, and for this purpose, sufficient and efficient staff should be maintained.

Regarding the construction work, enough has been said already in the preceeding chapters, and about maintanance work details are given in the following paras of the chapter.

2. Administrative Classification of Tanks :—For administrative purposes, tanks are classified in some States (for example in old Mysore) into two classes, major and minor. Those tanks which yield a revenue of Rs. 300/- and more are classed as major, and the remaining ones are classed as minor. Old Mysore contains, all told, 20,000 tanks, of which only 8000 tanks are major and the remaining twelve thousand are minor tanks.

3. Minor tanks :—These minor tanks are scattered all over the country and wherever there is some supply available, it is stored in a tank.

These are very useful for the cultivator and for the villager as well. The maintenance of these tanks is a problem and it is not yet solved completely.

4. Minor Reservoirs (Minor Tanks) :—In South India, in Madras, Mysore, and Bombay States there are a number of storage Reservoirs of small size, called tanks. These are used generally for growing rice or garden crops. Regarding distribution of water from these reservoirs, they are left to be managed by the cultivators themselves.

Advantages of minor reservoirs ;—(i) Isolated and small catchments which cannot be otherwise included in large schemes are utilised here.

(ii) The irrigable area being close to the tank and compact, the loss in transit is small and so the duty realised is high.

(iii) The reservoirs are generally located near a village and are thus convenient to the cultivators.

(iv) In a connected group especially in the case of anicuts, there is considerable amount of seepage from the *upper* to the lower ones, and thus water could be used again and again.

(v) The distribution system is managed by irrigators themselves and thus the administration is fairly efficient.

Disadvantages :—(i) The catchment areas being small, there is uncertainty of replenishment and supply to the lands.

(ii) The water spread area occupied by the tank is large, and the loss by evaporation is also large.

(iii) The works being more in number, the cost of maintenance becomes large, especially when they are liable to damage by floods.

(iv) As there are a good number of these tanks and they occupy a large area, the area of fertile land in the valleys for irrigation is diminished.

(v) There is extra cost involved for the proper supervision of maintenance works and general administration.

5. Maintenance :—(Tanks) : When a tank is constructed, it is completed in all respects to fulfil all its requirements and handed over to the Revenue Department for maintenance. To maintain the tank in good condition, the standard as sanctioned should be kept up, that is, the top width and the slopes, of the bund and the masonry works connected, etc. For this purpose, some guide stones are fixed on the bund, each of which indicates how the bund is to be maintained. Some of the important stones fixed on the tank bund are detailed below.

6. Standard Stones :—(i) *Grade Stone* :—Grade stones are fixed from the left hand side of the bund from its commencement, on the top and towards the water edge of the bund at 99 ft. centre to centre.

The length of the stone is roughly $1\frac{1}{2}$ to 2ft., width 9 in. and the thickness from 4 to 6 in. To fix the stones, pits are excavated and the bottom is filled with stone chips and gravel, or some concrete, and the grade stone is fixed on a bed slab to prevent settlement. The shoulder of the stone is fixed away from the water side and to the sanctioned top level of the bund (See Fig. 277).

GRADE STONE USED ON TANK BUND

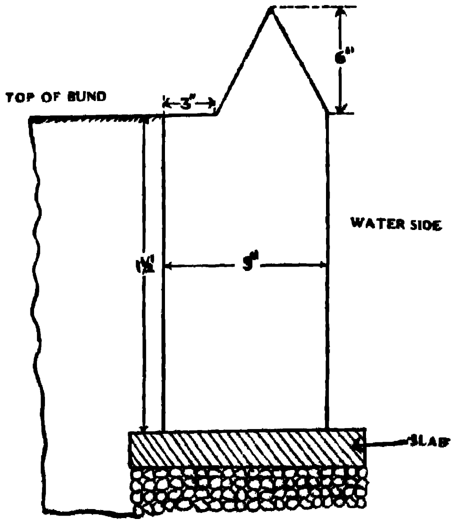


Fig. 277.

INDEX STONE USED ON TANK BUND

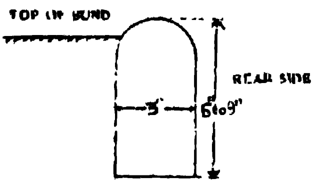


Fig. 278.

(ii) *Index Stone* :—The stone is fixed at the rear edge of the top of the bund. The distance between the index stone and the grade stone indicates the top width of the bund. The dimensions of index stone are roughly 9 in. in length, 6 to 9 in. in height, and 3 to 4 in. in thickness. (See Fig. 278).

(iii) *Register Number Stone* :—Each tank has got a register number of its own. This number is given separately for each Taluk, or Taluka, or Tahsil. This is to identify the tank at once. A tank register is maintained for each taluk, showing the register number of each tank in it. The length of the stone is about $1\frac{1}{2}$ to 2 ft. above ground level 15 to 18 in. in width and 4 to 6 in. in thickness. (Fig. 279).

REGISTER NUMBER STONE USED ON TANK BUNDS

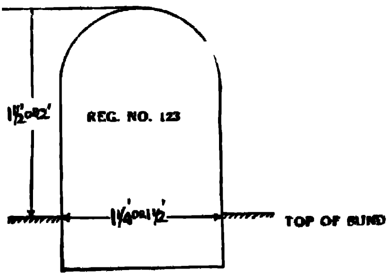


Fig. 279.

B. M. STONE USED AT FLANKS OF AN EARTHEN DAM

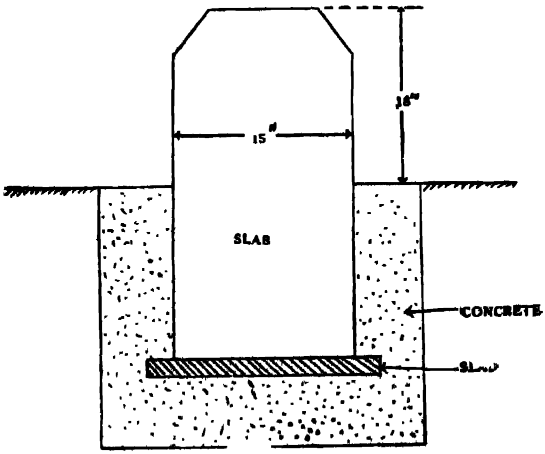


Fig. 280.

(iv) *Date stone*:—This stone is similar to the register number stone and in many cases, one single stone is used both for register number as well as date.

(v) *Bench Mark Stone*:—These are two in number, and one is fixed at each end of the bund. The stone must be very accurately fixed to the proper level, as on it depend all other levels. It should always be fixed on a good bed slab and in mortar. The size of the stone is 12 to 15 in. in width, 6 in. thick and 1 to 1½ ft. above the ground level. The top should be nicely chisled and be quite horizontal (See Fig. No. 280).

(vi) *Gauge Stone*:—This stone is intended to find the level of water in the tank or reservoir. This level may either be below the waste weir or above it; accordingly, the stone has got zero at the centre or middle height of the slab and 1-2-3 are read both upwards and downwards to show the level of the water in the tank. Vide Fig. 281. This stone is fixed at a place where the water is still or quiet. Each foot is divided into four parts (of 3 in. each) generally; but where more accuracy is required it is divided into ten equal parts.

GAUGE STONE USED IN STORAGE RESERVOIR

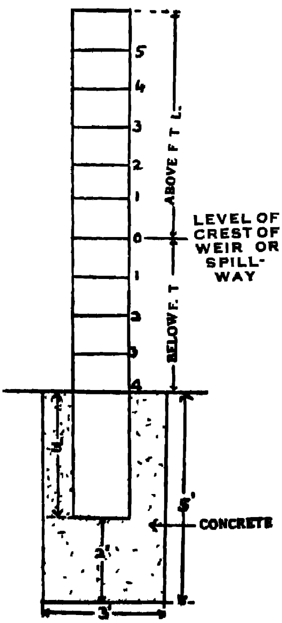


Fig. 281

(vii) *Survey and Constructive Details Stone*:—This stone is not generally used on all tank bunds; only in special cases where the estimate exceeds Rs. 50,000 or a lakh, it is provided. The height of the stone is about 4 to 5 ft. above the ground level, the width is 2½ to 3 ft.

7. Maintenance Work:—For this work, an amount alloted by Government annually under different heads:

(a) Irrigation Cess Fund,

(b) Acreage Cess Fund, and
(c) Contribution Fund.

(a) *Irrigation Cess Fund*:—Under this head, the works done are repairs to masonry work, such as, sluices, weirs, revetment of reservoirs and all cross drainage works in the channel.

(b) *Acreage Cess Fund*:—The works done under this head are the works that are usually to be done by the ryots or the cultivators of the irrigated area; but generally, as the ryots do not attend to them and allow the works to be neglected, the government levy a rate and recover the amount from the ryots to do these works, which are earthwork to bund wherever required, clearing silt in the channel, jungle clearing on the bund of the dam and on the channel bund, also weeds clearing in the channel bed, and turfing to the slopes of the embankment of the bund of the reservoir and of the channel.

(c) *Contribution Work*:—When ryots require any special work to be done, on their behalf, by Government, this amount is credited to the treasury and the amount is spent on doing the work required by the ryots.

8. Maintenance Works to be done by Ryots:—

(1) Maintaining the bund to standard section with required earthwork, etc.

(2) Removing and clearing the jungle growth on it.

(3) Providing or replacing turfing on the slopes of the embankment.

(4) When a sluice channel for distribution of water to the land exists, the silt clearance in the channel and also the upkeep of the bund of the channel, as detailed in (1), (2) and (3) above.

All the above works will have to be done by the cultivators under the tank.

9. Maintenance (or Repair) Works to be done by Government:—The dam requires in addition to the above, repairs and maintenance of the masonry works, such as sluice, weir and also the revetment. These works are done by the Government.

Irrigation works are subjected to leaks, slips and breaches. These, if they are of small magnitude, are attended to by the cultivators themselves, and if the amount involved is heavy,

then Government generally comes to the rescue of the ryots, and attends to

(a) Repairing the slips,

(b) Closing the leaks, and

(c) Filling up the breaches.

10. Slips in Earthen Dams :—*Causes* :— (1) A slip in an earthen embankment is due to the over-saturation of the down-stream side of the dam.

(2) If the dam is constructed of cohesive soil, when there is a sudden or rapid draw-down, it resuls in a slip of the front slope. Hence this should be made as flat as the situation permits.

(3) Since the soils in the slopes possess very little shearing strength, slips occur owing to the steepness of the downstream and upstream slopes. To avoid the same, the slopes should be made flat.

(4) Due to excessive percolation of moisture, the foundation becomes soft and sodden and since, in this condition, it cannot withstand the weight of the embankment, it slips. This can be overcome by compacting the soil properly.

(5) Slips may also be due to heavy rains on the downstream slope, and to waves on the upstream slope. This is prevented by pitching or stone revetment.

(6) Slips may sometimes be due to the soil being mostly of clay, and as a result of slips, smooth surfaces are formed round about the bank. The clay in such cases should be replaced by good gritty material (to avoid slip).

Prevention :—" Prevention is better than cure " is better applied in the case of an earthen dam. If the dam is constructed on a good and sound base or foundation with good materials and in proper duration of time (and not hurried up) and if there are free outlets to carry off the seepage, then cases of slips will be very few.

To overcome slips, the following points, will have to be considered :—

(1) The site of the base of the dam should be properly cleared by removing trees, bushes with roots and all sorts of rubbish.

(2) The base of the dam should be properly formed and constructed.

(3) Rough materials, like murum, should be provided at the outer toe of the embankment for proper drainage.

(4) At the base of the dam, cross drains and longitudinal drains should be provided for the outlet of seepage water.

Repair of Slip.—(5) When a slip occurs in an embankment, the slipped portion should be completely removed and even from the portion below the ground level, all loose and slushy stuff should be removed outside. The rough stone jelly drain, if it is choked up with earth, should also be removed outside, washed clean and rebuilt as per design. Then the site of the slipped portion should be stepped back with flat benching and with grips cut into them, so that the new earth might join well with the old. Fresh earth should be brought and placed layer by layer, slightly sloping towards the interior and rammed well. As there is moisture already in the base of the dam, only very necessary (and small) quantity of water should be used for compaction. After the earthwork is brought to the proper top level, the side slopes should be formed correctly to the sanctioned slope. At the rear toe, thick rough stone revetment should be built to a safe height to drain off any percolation water. Necessary base drains and lead-off drains should also be constructed at the toe.

11. Junctions in Embankments:—The junctions in embankments should be as few as possible, and when they are made, they should be made in easy gradients and for great lengths, and in addition, grips or forks should be cut into the sides so that the old and new may bond well. (See Fig. 282).

PLAN OF JUNCTION OF OLD AND NEW EARTHWORK SHOWING GRIPS OR FORKS

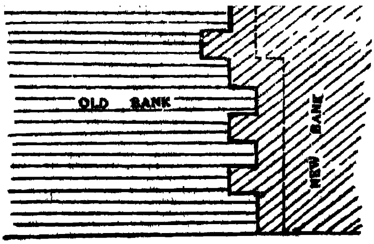


Fig. 282.

Whenever possible, the height or depth of junction of the embankment should not be made more than 20 ft. in one season.

When an old bund or bank is to be made larger in section, it is raised in height and consequently widened also, the old slopes should be cut into flat steps which should slightly slope towards the inside of the old section, as shown in Fig. 283.

JUNCTION OF OLD AND NEW EARTHWORK

Fig. 283.

12. Leakages:—*Causes of Leakages*: Openings which permit the free passage of water are created through earthen dams and these are called by the name leakages and are due to the following causes:

1. During the construction of the earthen dam there might be no proper bonding between the sucessive layers of the embankment.

2. The lowest layer of the dam might not have been properly bonded to the foundation.

3. When there are pipes used for outlets (sluices) if proper protection is not provided at places along the length of the pipe to obstruct the free flow of water, the water follows the exterior surface of the pipe.

Note:—Pressure conduits require special attention. They should generally be built inside concrete or they should be inserted in a masonry tunnel with free space around them.

They should be well supported on concrete bed throughout their length and also fully covered all round.

If supports are provided only just at the junction of the pipes, where the earthwork between the pipes settle, the pipe may give way, if it is not strong.

Again even if the pipe is strong enough to support the weight on its top, when the earthwork below the pipe shrinks, a passage is created between the bottom of the pipe and the bed for the free flow of water.

4. If the material used for the embankment is more of a pervious nature, water easily finds its way out.

5. Sometimes, a cavity or a tunnel is formed in the bund by burrowing animals.

13. Action to be taken in the case of leakages. If it is reported that a leak has occurred in an earthen embankment, the first duty of the person in charge is to see whether the water that escapes through the leak is clear or muddy. If it is clear water, there is no immediate danger; but if it is muddy water, action should be taken without delay. (Clear water means, that only water passes, from one side to the other; and muddy water indicates that along with water, the soil particles of the embankment are also being washed away). It should also be noted whether the leak is small or big.

The next operation is the correct location of the hole or leak so that it may be tackled correctly and the leak closed promptly. The passage of the leak from the front to the rear of the embankment is not always perpendicular to the centre line of the embankment, as the water takes the line of least resistance to flow out. This may be at any angle to it; hence if the leak is observed at the rear of the bund, one cannot be very sure that it begins just in front of it (at right angles to the embankment). This has to be verified and the correct position of the leak in front is to be fixed. This is done by one of the following methods :

(1) When a leak has occurred in the embankment and it is large and water rushes from front to rear and there is whirling action in front just above the leak, this should be observed and the point fixed.

(2) If the leak is small, and if there is no whirling action and tendency of water to flow towards the leak. Here if some heavy turf sods are thrown on the surface of water, near the approximate location of the leak, they are attracted towards the leak and come out at the rear. This method also fixes the correct position.

(3) In the case of leaks which cannot be located by either of the above methods. the following gives mostly accurate result :

At the point of the leak observed at the rear of the embankment, insert the end of a bellows of a blacksmith into the hole or

leak and pack it up with clay. Now put fire nearby, and work the bellows towards the water side and bubbles can be observed at the leak portion in front; thus the correct position of the leak is located. (See Fig. 284).

After fixing this (correct) position of the leak, the next operation is to stop the leak.

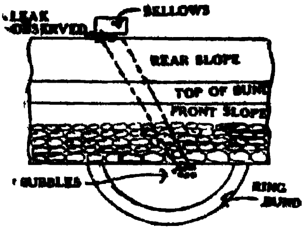


Fig. 284

If the leak is small, a diver is asked to go to the spot with balls of clay and turf and fill the hole in front with them and tamp it or push it inside the hole. After satisfying that the leak is stopped by this method, dump down a large quantity of earth in front and attend to the final and proper repairs after the rainy season is over.

If the leak is big, put a ring bund in front and bale out the water between the ring bund and main embankment. The leak portion is now visible. Remove any disturbed revetment and make the hole bigger by cutting steps to a length of 3 to 4 ft. inside. Fill up the hole with good clay and push it inside as far as it permits. The front portion should be packed up with good clay and the revetment rebuilt with proper jelly backing or small chips of stones.

At the rear also, the hole should be filled in, as much as possible with small chips of stone, and rough stone revetment built and a lead-off boulder drain constructed, to drain any water to the natural valley nearby.

14. Closing the Breach :—All the water should be baled out from the bottom of the breach portion; all silt and sand and vegetation met with and roots up to the very bottom, should be removed and excavation made into the original ground so as to reach an impervious soil.

The sides should be cut into steps, and all loose soil therein removed. Grips should also be cut at the bottom and sides, so

that good bond may be formed between the old and the new work. (See Figs. 282–283).

At the breach portion, where one cannot get good soil, always provide a puddle wall (except for shallow tanks). If rock is met with, in the centre of breach, a masonry core wall should always be provided. Sometimes both a core wall towards the front, and a puddle wall in the centre are provided. (See Fig. 285).

CLOSING THE BREACH IN AN EARTHEN DAM
SHOWING CORE WALL AND PUDDLE WALL

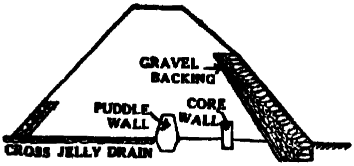


Fig. 285.

Earthwork in the breach portion should be carried on throughout the whole area and the bund raised with 6 in. to 9 in., layers in a concave section. (The ends being more raised, 3 in. to 6 in. higher than the centre) See Fig. 286. The consolidation should be very perfect. In fact, more water than is actually necessary should be used at the breach and the consolidation completed. (Consolidation by cattle is more satisfactory than by other methods).

BREACH FILLING IN AN EARTHEN DAM SHOWING
CONSTRUCTION.

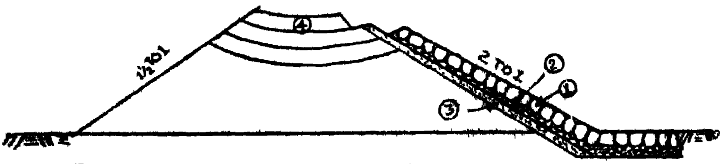
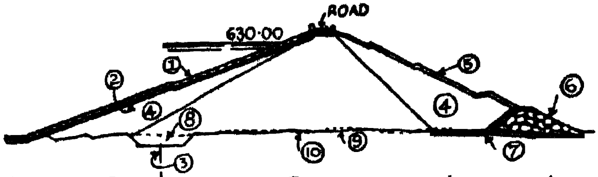


Fig. 286.

Use very good selected stuff for the front portion, that is, between the puddle wall and the revetment.

Make the section of the breach portion flatter than the adjoining portions. Instead of one uniform slope for the rear, two or three different slopes with horizontal berms are sometimes provided (at the rear) when the height is great. Vide Fig. 287.



① RIPRAP ② GRADED FILTER ③ GROUT HOLES 30' DEEP AT 20' CTRS
④ SEMIPERVIOUS SOIL ⑤ TURFING ⑥ ROCKFILL ⑦ FILTER BLANKET
⑧ CUTOFF TRENCH ⑨ STRIPPED SURFACE ⑩ ORIGINAL GROUND SURFACE

HIRAKUD PROJECT
EARTHEN DAM
SECTION

Fig. 287.

The construction of revetment work should also go on simultaneously with the bund. Its top should be about 2 to 3 ft. lower than that of the bund. The front toe of the revetment should be protected with a low platform. Behind the revetment, there should be jelly backing 1 ft. in thickness and gravel backing 9 in. to 12 in. thick. During construction, the top of the revetment should be lower than that of gravel backing and this gravel backing lower than the top of the earthwork proper. See Fig. 286. If there are pits nearby, the foundation of revetment should go 2 to 3 ft. lower than the bed of the pit.

The top of the bund at the breach should be made higher than at the sides (as sanctioned) to allow for settlement. (About 1/16 of the height is allowed for settlement of embankment of storage reservoir).

The top of the bund, the front slope up to the revetment and rear slope complete, should all be gravelled about 9 in. to 12 in. thick. This will prevent soaking of rain water and also scour in the slopes. These surfaces (except the centre portion on the top) should be turfed.

The scours in the bed in the front and the rear of the breach should be completely filled in, especially the rear portion, with good earth and well consolidated, and the top 2 ft. thickness over the old scour may be pitched with rough stones and well drained. (See Fig. 288).

BREACH FILLING PROTECTING THE SCOUR.

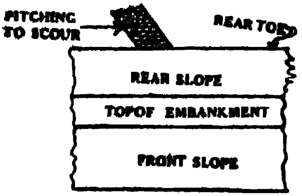


Fig. 288.

The base of an earthen dam should always be well drained. This should be specially so at the breach (or the river portion). For this purpose, both longitudinal and cross jelly drains should be provided for; also the rear revetment should be built, as a safety measure.

INVESTIGATION OF AN IRRIGATION PROJECT

1. **Reconnaissance** :—Before fixing up the site for a reservoir, a careful study of the Topo sheet (1 in. = 1 mile) should be made, as it will give a good idea about the suitability of any valley for storage and irrigation.

It should be ascertained if the water supply from the catchment area is sufficient and if there is enough good land accessible to the water supply. If these are available, i. e.

(a) Ample good land, fairly smooth and fertile, and

(b) An apparent water supply, with no obvious insurmountable difficulty, then surveying may be started to

(1) Measure the water supply, and

(2) Determine the cost of storing, controlling and bringing it to the land (to be irrigated).

A personal inspection of the whole area is however necessary and the following information should also be collected:—

(a) Area to be irrigated, (b) Nature of the soil, (c) Crop to be grown, (d) Population, (e) Possible canal location, (f) Probable difficulties in construction, (g) Communication, (h) Materials easily available.

2. **Approximate Estimate** :—From the topo sheet, the area of gross command is easily measured and leaving high ridges here and there, the commanded area is got; and even here, the whole

area cannot be irrigated because the soil may not suit the nature of the crop to be grown. Hence the actual cultivable area should be fixed.

Taking this area into consideration, and also the average duty of the crop, the total quantity to be stored in the reservoir should be calculated. This fixes the full supply level of the reservoir, and allowing for the flood discharge and the free board, the top level of the reservoir and thus the rough cost of the dam is known, and the canal and other subsidiary works are roughly estimated. The total cost arrived at should be calculated per acre of cultivable area and only when this is economical, the work should be taken up.

In any project work, preliminary investigations are first done and only when this is approved by higher authorities and found to be economical, the final investigations are taken up.

3. Preliminary Investigations:—In connection with this

(1) It should be seen, whether the site proposed is well adapted for the project.

(2) It is always a good policy, to select, in the first instance, two or more sites for the location of the project work.

(3) To get an idea of the cost of the work, all the required information should be got.

(4) The investigator should see personally every part of the site and should satisfy himself that it is well suited for the work.

(5) In addition to the above information, it is also advised to take photos of important features.

(6) Again for the location of the plant and the camp required for the staff and the workmen, the required area should be noted and it should be seen that this area is high above the HFL of the reservoir.

(7) It may become necessary in some cases to shift or divert the existing communications such as highways, railways, navigation canals, in the locality, and these should also be marked on a rough plan.

(8) For the construction work, materials such as, stone, lime, steel, cement, timber, etc. are required. It should be noted where these materials could be economically got.

(9) For the project work, some area of land would generally be required, and this should also be noted and marked on the plan.

(10) The accessibility to the site of work is an important item; and so it should be noted whether any communication line, such as railway or roadway should be newly laid or extended to the site of the work.

(11) Last and the most important of the items to be examined is the foundation. Here the materials that are met with in the foundation should be tested; and specimens of the different kinds of rock met with during the boring of a hole, or the excavation of a trial pit, should be taken and tested for safety. In addition to the above, the elevation or the R. L. of the rock surface, and the dip and direction and the quantity of over-burden should all be noted. As the foundation of any structure is important, especially in big irrigation works, an expert Geologist should be consulted in the matter.

4. Final Investigation:—When the preliminary investigation is completed and it is approved by the authorities concerned, final investigation work should be taken up. Here the relative merits of the different designs at the different sites as selected, should be compared and the best one adopted. The question of foundation also, regarding safety and cost, should be settled, before the final design is made. Details of the area of the land required for the construction, and the land submerged by the water-spread, should be correctly taken and proper amount allotted to it for acquisition, in consultation with the Revenue Department. As stated above, for the relocation of the communications (roads and railways) correct alignment should be made as approved.

Final location of all the details should be made in this investigation work and designs of detailed drawings should first be prepared and from this, a correct estimate of the cost of the work should be arrived at.

5. Water Supply:—In the case of large and important Irrigation Projects, where time admits, the discharge of the river should be measured throughout the year, at the site selected for the storage works and also at the site of the headworks of the canal.

Unless there is (1) a long record of stream-flow kept by Government, (2) or the water supply is from a big perennial river

measurements should begin, as soon as possible, and this should continue (to be safe) for several years; gauging stations should be established and continuous records maintained. One should not be misled by formulae for the assumption of the run-off from data of the rainfall, evaporation, etc. These should be got only from actual measurements.

The records of the rainfall in the valley and in the area under command, should be collected. New Rain Gauge stations should, if necessary, be installed in the cultivated area and from the above data, and also, the nature of the country the run-off assumed, and the discharge calculated. This should not also be taken as correct, but only taken as a guide to check the discharge of the stream.

When the water supply is to be stored in a reservoir,

(a) the existence, (b) the capacity, and (c) cost of storage must be investigated.

6. **Surveying**:—When surveys are commenced, B. M.'s (Bench Marks) should first be fixed and an arbitrary datum assumed for the B. M.'s, and as soon as possible, these should be connected up with the nearest M. S. L. datum and all the levels reduced to M. S. L.

Surveying Tanks :—A base line along the valley should be taken, and a series of levels should be run up the valley. Cross sections at intervals (say 1 to 2 miles) should be taken according to the magnitude of the project. B. M.'s should be fixed at the end of each of these cross sections. If the results are satisfactory, contours should be run and tied on to the cross sections, and to the base line.

Dam Site :—Other conditions being equally favourable, alternative dam sites should be selected, after thorough surveys and examination as regards

(a) Suitability of Foundations, etc. (b) Availability of materials.

Trial pits should be got excavated (to know the character of the foundation of the dam and the abutments) first at 1000 ft. apart and then at 500 ft. and then at closer distances, depending on sites found favourable.

The amount of sediment carried by the stream and its effect on the life of the reservoir should also be studied.

Waste Weir :—To ascertain the best site for a waste weir, close contouring and trial pits, up and down stream of the waste weir site, are necessary to find out the suitability for the foundation. These trial pits should go down into rock and bore holes should be driven at least 10 ft. into the rock, to ascertain if it is sound.

Compensation —When the F. S. L. (Full supply level i. e., the level at which the water in the tank will stand) has been more or less determined, the nature of the land to be submerged and the crops of these lands should be noted; also the houses and villages to be submerged should be valued and the population recorded. From this, the probable amount of compensation to be fixed, can be estimated.

Surveying Canals :—For a canal, trial lines may be run to test various alternatives. A topographical survey map of the country may be made, on which all alternatives may be examined before making any detailed survey. The alternatives such as, tunnel or open canal, flume or inverted syphon, etc., may be compared.

The preliminary canal line may be surveyed by running a rapid contour, falling at the rate of 8 in. to 24 in. per mile, according to the nature of the work. The canal line will start from the reservoir or the proposed headwork of the canal, to the tail end of the land commanded. At difficult places, namely, steep ground, big nallas, etc , a few alternative lines should be roughly levelled up, and sufficient information collected to enable proper alignment to be selected on a subsequent or second survey. The principal purpose of the preliminary survey, is to ascertain the area likely to be commanded—a matter of great importance.

After the preliminary survey is over, if the investigation gives satisfactory results, then the second or the final adjustment is to be set off in straight lines.

Station :—The selection of the stations is based on the following considerations :—

(a) Straight lines should be made as long as possible.

(b) The included angle between the station lines should be made as large as possible, so as to get easy and flat curves.

(c) The country below the station last fixed, and the area now being fixed should be studied, and the new station be selected so that the line joining the two stations, will not give a canal, either too deep in cutting or too high in embankment.

(d) In crossing nallas and streams, a right-angled crossing, as far as possible, should be adopted and such a point selected in the bed for the crossing as will give a sufficient headway for the masonry cross-drainage works.

Having fixed the station points on the trial line, the included angles can be ascertained by Theodolite Survey and the curves laid out.

Detailed levelling can then be conducted on the final line (for longitudinal section, levels should be taken at every 200 to 300 ft. to show the inequalities of the ground surface.)

Cross section should be taken at every furlong or most often at difficult places and at every station point or at least 2 or 3 stations and should be carried to an adequate distance on either side to know the transverse slope of the ground. The staff readings at cross sections should be taken at 20 to 30 ft., so that a very close approximation to the desired level for the central line, may be obtained.

B. M.'s should be fixed on permanent structures at every mile. A note should be made of the soil, and of the features of the country passed through. Trial pits should be made in the canal line about 8 in a mile, so as to ascertain the nature of the soil, and the proposed canal bed; also trial pits are necessary where masonry works are to come, at nalla crossings, etc.

While the work described above is in progress, an independent set of check-levels should be run on a direct line connecting the permanent B. M.s. so to discover any errors that might have occurred.

The alignment is to be set out for the main canal and the branches only, and the surveys for the minors and the distributaries need not be made in the first estimate of the project.

7. Designs and Drawings. Complete drawings, on the sanctioned and approved scales should be got prepared of the following :—

(1) The site plan of the works. (2) The tank or reservoir. (3) The area commanded, showing the soil survey also. (4) The storage and the dam. (5) Headworks of the canal. (6) The canal proper. (7) The masonry works on the canal.

Soil Survey:—A detailed survey of the land to be irrigated is to be made before actual construction is begun, to determine the area of good land available and the fertility of the soil of the land. This information is very important, but generally neglected.

The classification of the soil —good, average, bad (barren, waterlogged, etc.,)—of the land to be irrigated should be shown on a tracing from the R. S. map of the village.

Estimates:—Estimates should then be prepared for the works:-(1) The dam, (2) The waste weir, and (3) The canal, in detail.

(1) *Dam*:—The foundation is very important. It should go 5 to 10 ft. below sound rock level. Soft soil, treacherous earth and all vegetable growth should be carefully removed from the foundation and proper drainage under the dam should also be provided for. The dam may be of masonry or earth work, as per requirements.

(2) *Canal*:—In estimating for the canal excavation and embankment for large projects, for preliminary survey, the depth or height should be taken to the nearest 6 in. and the distances at 200 to 400 ft. apart. But when the final alignment and small projects are taken up, the levels should be taken about 100 ft. apart, and the depth more accurately.

General:—Provision should also be made for the construction of stores, office sheds, roads and sanitary arrangements. The labour and materials available for the work should be very carefully considered and the rates for the work and probable time for the construction should be fixed up and estimated.

At any stage of investigation, temporary estimates of cost should be made, so that the unnecessary expenses may be avoided. Tendency to underestimate must be strictly avoided.

This is done to reduce cost, as it should be, but

(a) inaccurate data are given and this should be discouraged. (If the inaccurate data are made public and ridiculed, then explanation comes, of unusual "difficulties—abnormal conditions—storms —accidents—or the stupid blunders of some one else").

(b) Or sometimes, some items are omitted, like plant-cost or over-head charges—preliminary works, etc. These should be provided for in the estimate. Rates are taken for some favourable items only, thus omitting expensive delays, repairs or other contingencies. Hence "the path of estimate is beset with pitfalls," which one must be careful to avoid.

8. Report:—Every report should be set forth in a clear and concise form. All data on which conclusions are based should be given and appended to the report. Information regarding the following details should be included in the report:—

(a) Population of the village. (b) Nature of the land, crops grown thereon and the assessment. (c) Existing system of cultivation. (d) Liability of the area to famine. (e) Existing and proposed road and railway lines. (f) Markets for the sale of produce. (g) Results or effects of irrigation. (h) Liability of the soil to waterlogging. (i) Labour and materials available, so that the rates for different items of work may be fixed from them and the probable time for the construction and completion of the work calculated.

Information from similar projects in the locality may be got and used. The opinion of the Revenue and Agricultural departments will have to be got and embodied in the report; also the financial forecast will have to be approved by the Revenue Department.

The report should be drawn up in a logical order and each subject should be discussed under an appropriate heading. The ability to prepare a good report is one of the most important assets of the successful career of an Engineer.

The following headings may be taken as a guide in framing the reports and will have to be modified to suit any particular project.

(A) *Introduction*:—There should be a few introductory sections leading up to the main body of the report. This is to include, the authority for the report, the data on which the report is based, the Engineer's opinion, the period covered by investigation and general information up to date.

(B) *Location*:—The correct location of the project is to be described, and the Road and Railway facilities should be noted, and a map showing all these should be appended.

(C) *History* :—A general description of the project should first be given. Then details such as, topographical features, the project area and a detailed outline of the scheme. A general map (on a larger scale) showing the area benefited, etc., should be enclosed.

(D) *Water Supply* :—The success of any project depends upon good water-supply available, and as such, the subject should be dealt with in detail. Water-supply available for the project should be arrived at from a study of the drainage area of the stream flow and other hydrological data, allowing any deductions for the priority for water. The run-off from the area should be discussed in detail and all precipitation records and other data necessary to support the conclusions, should be given. The period of irrigation and the water-supply necessary for the crop should be known and thus the water-supply required for the project determined, taking into consideration the area to be irrigated and the losses in conveying.

(E) *Climate, Soil and Crops* :—The growth of any crop depends upon the climate of the place and this subject should be dealt with in detail. The soil formation in the whole irrigated area should be dealt with. If this varies greatly, a soil map should be enclosed. The kind, yield and value of crops to be grown should be noted in the report. The market for the crops and transportation facilities and thus the charges incurred should be discussed and the net return from the crops calculated.

(F) *Financial Aspect* :—An estimate of the whole project should be made on a liberal basis, so that ample allowance is made for all contingencies. An estimate of the actual cost of the project work should be made, also an enstimate of the annual income from the project. From this, the net return on the proposed outlay will have to be made to see how the project pays.

26th Annual Meeting of the Central Board of Irrigation Power at Delhi on 8 Nov. 1955

For any engineering work, worth mentioning, Surveys (a preliminary survey and investigation for preparing alternative estimates) are always necessary and important; and any reasonable amount spent on this account is worth it. This statement is endorsed or affirmed by the Deputy Chairman of the Planning

Commission of India, Shree V. T. Krishnamachari, recently during the Twenty-sixth annual meeting of the Central Board of Irrigation and Power, in Delhi, on 8th November 1955. He has said: " A careful survey should be made by engineers in every state, of the possibility of bringing more land under cultivation, through large and medium irrigation projects, and also minor irrigation projects" and the Government has provided a decent amount of 5·5 crores for this purpose, in the Second Five Year Plan.

Finance for the Project: He has also suggested that for financing irrigation projects in India

(1) Area benefited by canal irrigation should pay a betterment levy.

(2) Irrigation must be regarded as a commercial service, which should pay for itself.

Development of Irrigation: *Major*: The area secured for irrigation before independence was 26½ million acres. Under the First Five Year Plan it was 23·5 million acres at a cost of Rs. 708 crores of which 7 million acres are developed during this period itself as against the target of 5·7 million acres. Area to be developed under irrigation during the Second Five Year Plan is 13·8 million acres at a cost of Rs. 363 crores, and 15 million acres are proposed to be developed during this (Second Plan) period, and the remining area of 15·3 million acres will be developed afterwards.

Regarding *Minor Irrigation Projects*, the proposal is 9 million acres to be developed in the First Five Year Plan and 9 or 10 million acres during the Second Five Year Plan.

For the development of irrigation, it is not merely the area developed that is important, but the productive capacity of the land. Hence it is now proposed, to appoint a Committee to go into the question of increasing the productivity of the areas which have already been brought under perennial irrigation. With this idea, the President of the Conference, Shree Desraj Mehta, has said: " Self sufficiency in food lay in stepping up production."

For this purpose, good and efficient means should be used to irrigate the land and get more production from an area, without wasting water " indifferently and haphazardly." He has stressed the need for a very efficient and well organised Irrigation Service of Engineers.

9. General Notes on Project Drawing Work :— The drawing paper should be single elephant drawing paper 22 × 27 and may be used for general outline plan.

The scales to be used for the different drawings are noted below :—

(1) Dam or bund, water spread, wet cultivation,— 4 chains = 1 in.

(2) Datailed survey of weir or anicut, aqueduct, etc., 1 in. = 66 ft.

(3) Longitudinal section of bund or dam, weir, channel

horizontal	4 chains = 1 in.
vertical	40 ft. = 1 in.

(4) Bed sections, one along the bund and the other along the valley.

Horizontal	4 chains = 1 in.
Vertical	1 in. = 20 ft.

(5)	Masonry works	1 in. = 10 ft.
	Details of masonry works	1 in. = 5 ft.
(6)	Other sections	1 in. = 20 ft.

General :—(1) In every plan the north line should be shown invariably.

(2) In taking cross sections, all distances should be taken horizontally.

(3) The name of the individual taking the survey and levels, together with the date, should be noted on the plan.

(4) The name of the draftsman along with the date of plotting should also be noted on the plan.

(5) The amount of estimate should be noted.

(6) The signature of the officer who scrutinises the estimate should also be taken along with the date.

10. Important Enclosures of a Project for a Major Tank :—

(A) *Reports, Estimate and Statements* :—

(1) Report.

(2) Estimate.

(3) Survey and constructive details.

(4) Atchkat (ayakat) statement.

(5) Statement of lands to be acquired for
   (i) Submersion.
   (ii) Channel.
(6) Calculation sheet for waste weir
(7) Statement showing revenue under the tank.

(B) *Plans.*

(1) Topo sheet extract.
(2) Extract from revenue survey map.
(3) Sluice—Longitudinal section, cross section and plan.
(4) Weir—(a) Longitudinal section, Cross section and plan.
   (b) Block level plan.
(5) Bond—Longitudinal section and cross sections.
(6) Contour plan of the tank or reservoir.

11. Financial Aspect : This is an important aspect of any irrigation work. All irrigation projects should benefit the people and the country and should also be remunerative; if not, they must at least pay a good return. During the British Rule, the Government was caring more for a good percentage of return on the money spent (that is not less than 3 to 4%) and so a good many projects were simply shelved. But now, after we got independence, it is the policy of the Government to get on with all irrigation projects, whether paying or not, for the welfare and the prosperity of the people. Accordingly a good number of projects big or small, which will benefit the country, is taken up (under the five year plan).

From the works constructed, the people are benefited; and in return they should naturally pay something to the Government. Accordingly, taxes are recovered from them and the amount realised is adjusted, partly towards the original cost of the work and partly towards the annual maintenance.

IRRIGATION RATES

The system of levying taxes is dealt with below.

1. Introduction :—The object of levying watercharge and land tax is to use the amount collected for the improvement and maintenance of the works. This water rate will induce the

cultivator to use the water economically; that is the water will not be unnecessarily wasted by the cultivator, since he has to pay for it.

Water is supplied to the cultivators by the Government, and for this (irrigation) water, they will have to pay charges, and this is called *irrigation revenue* or *irrigation assessment*. Irrigation revenue also includes the land tax, which the owner of the land has to pay in addition (to water rate).

The water rate is received either from the owner or the cultivator for the water supplied to the land. The (irrigation) water rates levied are of three classes:

(1) Volumetric Rate, (2) Crop Rate, and (3) Seasonal Rate.

2. Volumetric Rate :—This rate is fixed according to the volume of irrigation water supplied to the cultivator. By this method water is used economically. This method is not much in use in India, since it is not liked by the Indian cultivators.

The volume of water supplied is measured by passing the water through either modular outlets or Venturimeters.

Modular outlets are being used in India only in some places. This type can be used only where volumetric rates are levied.

Venturimeter is not adaptable, since it is costly and there is much loss of head at the head of the water course, due to the water carrying large proportions of silt.

Disadvantages of Volumetric Method:—The cultivators at the tail end of the water-course will receive a less amount of water than what is actually supplied at the head of the water-course, since there will be loss of water during transmission. But the irrigators at the tail end of the water-course will have to pay the charge for the water supplied to them at the head of the channel and not for the water they actually receive. Only the cultivators at the head of the water-course will be benefited.

3. Crop Rate :—This is much in common use in India, since this is the rate charged not for the whole land but for the area where the crop can be grown satisfactorily.

The land will be investigated by an officer (of P. W. D. or Revenue Department) appointed for the purpose, who will measure

the area on which crops would grow, and the crop rate is charged on that area only.

The crop rate varies from canal to canal and from State to State, also. The crop rate depends upon

(i) the nature of the crop itself, and

(ii) the value of the crop (in the market).

If the crop consumes more water for its growth, the crop rate will also be high. If the market value of the crop is great, its crop rate will also be great. When the irrigation is done by lift method, the crop rate will be less (nearly half).

4. **Seasonal Rate** :—As the name itself suggests, this is the rate charged to the cultivator for a certain season. The rate depends upon the different crops that can be grown in the season and the water consumed by them; as such, the seasonal rate varies from season to season.

N. B. :—Seasonal rates are adopted in Burma, but are more complex than usual.

5. **Composite Rate** :—Eoth the land tax and the irrigation water-rate are collected together in some States of India. This combined charge is called Composite Rate. This system of levying charges is the most simple and the best one where the irrigated area is subject to only slight changes.

Generally, the water rates in Bombay are greater than in any other part of India, since the crops grown here fetch more value per acre than the crops grown in other States.

It is seen that in California, U. S. A., the water rates are very high, because the cultivators are supplied water by pumping, and this water is distributed among them, very carefully and economically.

Questions

1. Give in a nutshell a brief outline of the various points which you would consider before deciding on drawing up an irrigation project.

(A. M. I. E., May 53 and Nove. 55)

2. A major tank is to be constructed across a stream to irrigate about one thousand acres. Explain in detail the data you

would collect and the plans you would prepare for the irrigation project.

(Mysore Univ. Sept 50)

3. You are asked to frame irrigation scheme for a district in the Bombay Deccan which will help to meet the existing food shortage in the country whithin a period of three years. State the considerations that would guide you in framing such schemes and investigations you would carry out for the purpose.

(Bom. Univ. B. E. April, 1950)

4. Give a brief outline of the field and office work required for investigation and submitting preliminary report for the construction of a storage reservoir across a valley. Mention the various estimates and plants that are required for the purpose.

(Bom. Univ. B. E. Apr. 1953)

5. Distinguish clearly between Crop rate and Volumetric rate. **(Gujarat Univ. April 1953, S. E. Nov. 53)**

(Department of Technical Education Bombay April 54)

6. What are (i) Composite rates and (ii) Volumetric rates ?

(Poona Univ. Oct. 52 B. E.)

7. On what principles are irrigation rates fixed ? What are the common methods of assessment of rates under irrigation and which one do you consider suitable for general economy ?

(Poona University April 54 B. E.)

8. What are crop rates and seasonal water rates ?

(Poona 52)

How would you conduct the necessary surveys for aligning the main canal from a reservoir as a contour canal ? What additional information would you collect during the above surveys, so that an intelligent finalisation of its location is possible in the design office.

(Mys. Univ. B. E. Civil April 1957.)

What are (i) Crop Rates, (ii) Block Rates, (iii) Volumetric Rates and (iv) Seasonal Rates ?

(Poona Univ. B. E. Civil. April 1957)

Write short notes on crop rates and composite rates.

(Gujarat Univ. April 1957)

CHAPTER XX

ARCHITECTURE

Architecture is the expression of the ideas or the design of the engineer in a concrete form.

Unity :—Presence of the quality of unity in architecture is important. Unity is the expression of the requirements of design. It is the composition to be arranged into a single unit or group of units. The expression of this idea should not be sacrificed in architecture.

Procedure to get best results :—To get the best results in architecture, the following points should be strictly followed,

1. Have a programme, well analysed.

2. Get details of all requirements and consider how best to provide the same, and solve it or work it out actually.

3. Read the programme well and get an idea of
   (a) The character, quality and type of the structure.
   (b) The site, surroundings and access to it.

4. The programme should be well written out first, and the difficulties, if any, should be noted. All the requirements should be classified as major and minor, then they should be put on the plan and studied how they appear there (on the plan). The main element should be located at the focal point and the secondary ones should be filled in later on.

The four principles governing an architectural design are

1. Sincerity.
2. Propriety.
3. Style.

and 4. Scale.

1. Sincerity :—Many engineers, feeling that a correct design for carrying the loads, providing for stresses or the taking care of the utilitarian features is the only thing requisite, stop short of the necessary artistic arrangements of the component parts of the structure and the use of some simple but appropriate decoration to make the design truly artistic.

2. Propriety :—A structure, if properly designed to be architecturally beautiful should have symmetry or balance or harmony among its component parts and with its surroundings. By propriety it is meant that the structure should show the bare truth and the details in it should not appear extraneous.

3. Style :—The style should be of proper proportions. It should harmonise with the other details of the structure. It should help forming an entirely pleasing ensemble.

4. Scale :—This is a feature of proportion. The scale is determined by the economic proportion. But these proportions should not be fixed at such extremes as might produce an unpleasing result.

One need not care about the style of architecture. What is required is *Proportion and harmonisation* with other details and with the structure, so that it would give an entirely pleasing general effect.

Ornamentation :—The amount and kind of ornamentation to be adopted in any engineering structure is as important as the determination of the form of the structure itself. Even when the design is conceived carefully and worked out in its fundamental features and the outline and the ensemble made pleasing, all this may be ruined by the lack or type of ornamentation used, or sometimes by the over-doing of this ornamentation. All ornamentations of a dam should be bold and massive.

Aesthetic Engineering Design :—The basic principles of any structure, when it is to be considered artistic or to have any pretention to beauty are 1. Simplicity, 2. Symmetry, 3. Harmony, and 4. Proportion.

1. *Simplicity* :—The structure should be outspoken or truth-telling. There should be no covering by another material to hide the interior one. There should be no frivolous or inappropriate details. The design should strictly adhere to the required features.

2. *Symmetry* :—This is essential to an artistic and pleasing design.

3. *Harmony*:—There should be harmony :

(a) Between substance and substance.

(b) Between various component parts of the system.

(b) Between the utilitarian features and the ornamental details.

(d) There should be complete harmony with the surroundings and the structures in the vicinity.

4. *Proportion* :—The proportion of the details employed for ornamentation is a distinct feature of the design; and to be harmonions they must be of the correct type properly applied, and of right proportions. The proportion may be said to be correct, when the entire structure is most pleasing to the trained eye.

The conception of the design of a work of engineering architecture must be based on :

(i) The use to which it is to be put.

(ii) The amount available for construction.

(iii) The materials which are to be used.

(iv) A visualisation of the location or the environment.

The material of which an engineering work is to be constructed is very largely the governing factor in its artistic treatment and what might be a very appropriate design for one kind of material would be very inappropriate for some other material.

The great dam of masonry is in as appropriate a setting in a mountain gorge as are the pyramids in the vastness of the desert

The successful designer must have the imagination to conceive how his structure will look and should design it to match with the surroundings.

DAM ARCHITECTURE

Dam Architecture :—The dam architecture is generally neglected. This is specially so in private enterprises which are mostly in vogue in America, where when a private party takes up the construction of a dam, it will not warrant the outlay required for aesthetic treatment.

The dam, in addition to serving the useful purpose of storing water, should also give a monumental expression of the magnitude and importance of the work. As such the following should be adopted.

(1) The downstream face of the dam should be covered with a layer of granite masonry, arranged in panels, and surmounted by continuous cut-stone cornice.

(2) The facing should exhibit extensive landscape work; for example, terraces, ramps and artificial pools, (all of these impart a pleasing appearance to the structure).

(3) The style of ornamentatión should be in keeping with the dignity of the structure.

(4) The appropriate treatment should depend to a large extent upon the appearance of the neighbouring landscape. The type of architecture should be in keeping with the surrounding area.

(5) The character of the site of the dam should also be taken into consideration. For example, if it is rugged, a simple massive and dignified treatment is necessary.

If the dam is of the type of a solid gravity dam, the weight of coping pilasters at the top are effective in adding to the stability.

If the dam is of a hollow type, the weight of water and not of masonry, offers the main resistance to failure, so that a greater expenditure is required to these embellishments.

If the dam is an arched one, an increased thickness at its top, in the form of copings or cornices, adds materially to the stiffness of the arch.

(6) The structure should be made as water-tight as possible, so that a pleasing appearance may be given to it. If not, even a slight leak will result, not only in discolouration of the downstream face, but it will also pave the way for the deposit of laitance, which is even more objectionable.

Examples:—Taking the T. V. A. Scheme into consideration, an idea of its architecture and its development may be had from the following notes.

One can find here, the architecture on a scale dictated by the eye.

Norris Dam:—In this dam, the architecture is very magnificent and dramatic. The mechanical and perspective has been humanised by the architect. Each power house has a visitor's building (solid and box-like) because of its relation to the main structure. The modern eye can appreciate the beauty in the machinery, better than in the fine arts. The purpose behind the machine and the appropriate form clearly express the ultimate purpose. The Engineer and the Architect have combined the

machines and the structures into a coherent design of great beauty and sensibility. The masonry is conceived as a part of the whole composition.

The two points that strike one most forcibly in the T. V. A. are

(i) The way in which the dams and power-houses have been designed in relation to their surroundings, so that they form a unity with the landscape and enhance the interest and beauty, instead of standing out, like so many ninteenth century utilitarian constructions, in gross and defiant conflict with the natural environments.

(ii) Attention everywhere paid to good design of details, handrails, lighting standards, gates and doors, and all the fittings and furnishings, etc.

For example,

(a) The transformers and switchyard, are not applied to a predetermined structure, they are part of it.

(b) The cantilevered over-head gantry-crane for moving heavy equipment from barges, is part of the architecture, as vital and telling, as the generator hall over which it runs.

(c) The huge pulleys on the overhead gantry are detailed, with as much care and precision as a metope on a Greak Temple. Like the metope, they are a part in harmony with the whole; and the whole is an architecture which can be appreciated and enjoyed by all.

Details :—Every detail reflects the strength of the complete structure of which it is a part. It is in the design of details that the architect, as the great humaniser, has played a magnificient role. From a major element to a minor, one finds the same meticulous care, taken in the choice of the material and the simplification of form. Once the element is reduced to its final shape, it becomes a standard for the particular job. The permanent nature of the construction is firmly reflected in the substantial material used; for example, aluminium, terrazzo, tiles and marble: and in exterior work, concrete and masonry. Nowhere are the details used which do not contribute to the one major effect. They are sensible and impersonal. Unconsciously, through them, the whole architectural effect is registered in the mind. They give scale and meaning, as they are purposeful and mostly related to the more normal human needs.

Most of the dams are located in natural pictorial surroundings and some of them are made artificially so.

Bhakra Dam. This will be one of the Engineering marvels when completed. It is the world's highest (680 feet) straight-gravity dam. But, if other types of dams are also taken into consideration, it is the second highest, the first being Hoover Dam in America, being 726 feet in height. "It rises majestically to a good height and is picturesqusly located at the entrance to the steep and narrow gorge in the last spurs of the Shiwalik Range of the Himalayas. The dam partakes of the majesty and might of the Himalayas."

Behind the Bhakra gorge, "the mountains form a kind of natural bowl of about 63 sq. miles" and all the water flowing uselessly to the Arabian sea will be impounded here, and about ten million acres will get the benefit of its waters.

Mayurakshi (West Bengal). The dam is situated at the precipitous height on the Rajmahal Hill range. In the impounded water here, "the bushy tops of good many trees and high mounds submerged are visible in the placid, brickish-red, liquid surface, giving them the appearance of so many velvety green saucers floating on it. When water is allowed through the sluices, it looks like a lively sprawling giant gushing water down in the fields. "The water stands brimful, placidly reclining against the man-made masonry dam, that will stand for generations to come as a living monument to the technical skill of the engineers displayed in raising the edifice on the ruins of the haunted memory of the bygone colonialdom."

Sarada Barrage (U. P.). A 2000 feet barrage over the Sarada river at Banbassa in Naini Tal District U. P. is a magnificent spectacle and it is one of the biggest in India. The barrage is provided with thirty four huge steel gates of 50′ × 11′ each. These gates are 12 tons in weight, and are operated so smoothly in well-oiled gears, that they can be managed by merely two men.

"The Sarada river emerges at Banbassa from the beautiful Kumaon Hills at the foot of the Mount Kailas, bearing the melted snow of the Himalayas".

Krishnarajasagara Dam (Mysore). The dam is situated about ten miles from Mysore and is 130 feet in height and 1· mile

and 6 furlongs in length. A nice artificial lake with a water spread of about 50 miles is formed.

"Putting into full use the natural contours of the terrain and abundance of water provided by the reservoir Sir. Mirza M. Ismail, when he was Dewan of Mysore, with his characteristic drive turned what were once barren slopes on either bank of the river, into terraces of beautiful loveliness where hundreds of fountains play perennially in myriad shapes." These Brindavan Gardens are formed very luxuriantly and are provided with delightfully shaded walks and fountains to match with the nature all round. The gardens produce a heavenly effect with various colours, especially during nights when the lights are on.

CHAPTER XXI

RIVER VALLEY PROJECTS

River valley projects are dealt with in detail Vide Page 116 to 139 of "Solutions to Problems in Irrigation Engineering" by the authors. Most of these projects are pushed through to completion (nearly) in the 1st Five Year Plan, and the progress so far made here are quite satisfactory and are well known to many of the readers; yet the progress on some of the works is noted below for information.

MULTIPURPOSE PROJECTS

1. **Bhakra-Nangal Project**.—This is given the highest priority of all the Multipurpose projects. This project is "History in the making".

Details of the project are given in page 126 of the book, "Solutions to Problems in Irrigation Engineering" by the Authors.

The Nangal Dam and canals have been completed. The canal was executed mostly by manual labour. (For the construction of the Bhakra dam concreting is being done by highly mechanised work.

The Bhakra canal system was opened by our Prime Minister Sri. Pandit Jawaharlal Nehru on 8th of July 1954. It was inaugurated as "the world's biggest irrigation canal network, which could irrigate 6 million acres of sun-scorched dry lands of the Punjab, *Pepsu* and Rajasthan states. It is a living witness of the growing strength of the nation and its determination to go ahead."

For the inauguration, Pandit Nehruji pressed a button on an electric dash board and one of the newly painted black sluice gates on the Nangal Hydel channel was slowly lifted, releasing a gush of steel-grey water of the Sutlej into the canal, Panditji declared "on your behalf and on my own behalf, I dedicate the Bhakra-Nangal work to the good of the Indian People, Jai Hind".

Details of the Project:—Reservoir capacity 7·4 million acre ft.

Dead storage 1·7 million acre ft.

Quantity used for irrigation 5·725 million acre ft.

Dam:—Top width of dam 30 ft. (for a motor road).

Canal System 2900 miles

The length of the dam at top is 1700 ft., and at bottom 325 ft.

The width at the base is 625 ft. and at special places it is 1310 ft.

Spillway:—The central spillway is of the over-flow type with a length of 260 ft. It is provided with four steel radial gates each 50 ft. long and 37·5 ft. high., which can discharge about 4 lakhs of cusecs.

Irrigation Outlets:—These are constructed of two tiers of outlets with 10 in each tier, at R. L. 1320 and 1420, with a capacity of 106,000 cusecs.

Power:—Two power houses are located at the dam site and will be started each with a capacity of 19,000 kilowatts. The total power proposed to be generated is 10·44 lakhs kilowatts and 6·21 lakhs firm power to be generated in two stages.

Organisation:—The organisation of the Bhakra Nangal scheme is worth copying. It consists of the following:—

(1) Designing Directorate.

(2) Construction and Plant-design Directorate.

(3) Directorate of Inspection and Control, which looks after

 (a) Drawings and details of the dam

 (b) Execution of the work and installation of the plants

 (c) Proper control to see that the work is executed as per specifications and design.

Finance:—The whole amount required for the work, is financed from the Centre.

Board of Control:—This consists of representatives of the Central Government and also representatives of the States concerned, namely the Punjab, Rajasthan, and Himachal-Pradesh.

Establishment:—The whole work is managed by efficient engineers, technicians, and other workmen. In addition to the above, there is a Chief American Engineer, Mr. Slocombe, with forty other American Engineers, who are training the Indian engineers in the work.

Dam Construction:—The location of the plant for concrete is about four miles from the dam proper. For the transporation of the materials, belt conveyors, cantilever-cranes running on trestles, and revolving cranes will be used. The concrete batching and mixing plant will be located at different levels, for pushing through the work expeditiously. As the water in the river Sutlej is very cool (being ice-melted water coming from the Himalayas) the concrete is cooled to 43°F and for this purpose, a cooling plant is installed, which will cool the water to 35° to mix the concrete. The strength of the concrete is tested for different pressures (due to the head) in the research laboratories at the workspot.

For the continuous flow of materials, the required railway lines and roads have been laid at different levels. A decent workshop at Nangal and smaller ones at the dam site and other places have also been constructed.

The Nangal dam and the main canal of 700 miles in length and the distributary of 2100 miles are all completed. The Power House at Ganguwal was put into service in Jan. 1955, and that at Kotla was inaugurated in June 1956. The Ganguwal and the Kotla Power Houses have got a potential of 3 units each of 24000 KW; but for the present, only 2 units have been installed in each.

1. **Dam Power Houses** :—On either side of the gorge of the river, there is one power house situated. The tunnels for the pen-stocks have been completed through the sides of the gorge. In each power house, it is proposed to install 90,000 Kws, 5 units on the left side and 4 on the right side.

The construction of the Bhakra Dam proper is in full swing. The work of the giant concrete dam began in November 1956, when our Prime Minister laid the first bucket of concrete and it is hoped that in three years the 700 ft. high dam will be completed.

For the work of construction of the dam proper, very high cofferdams had to be constructed in the river gorge; the upstream one being 215 ft. high and the downstream one 135 ft. high.

Before the concrete was laid the foundation had to be treated to be perfectly safe. Hence the process of grouting, i. e., pumping cement slurry under pressure into all the visible and invisible cracks, crevices, fissures, etc. in the hill side had to be attended to.

" Never before in the history of dam-construction was grouting on such a huge scale adopted ".

The concreting is being done by *mechanised process*, as the work is to be completed in *record time*. It is said that if the old hackneyed type of laying the concrete were adopted, it would take 30 years, whereas with the present mode of mechanised process, it would take only 3 years. The dam is scheduled to be completed in 1959–60.

The planning has been so adjusted with the construction, that the benefits could be reaped by the people simultaneously with the progress of work.

During 1954, irrigation was extended to 3 lakhs acres and this rose to 10 lakhs in 1955, and it is further increased to 18 lakhs in 1956–57.

The original estimate in 1946 was 75 crores and it is now revised to 173·5 crores (this may not be final). The expense up to November 1956 was 129·06 crores. The revision is due not only to the increase in the rates but also to additional works found necessary. The work is propsed to be completed slightly in advance of the schedule. When completed the Bhakra dam will be " one of the marvels of Engineering of all times " or, as Sri. S. K. Patil recently called it, " a miracle of Engineering ".

2. Damodar Valley Corporation :—This project is developed on the same lines as the Tennessee Valley of America. It consists of the construction of four dams across the river Damodar and its tributaries namely, Tilya-Konar-Maithon and Panchet Hill, a barrage and the irrigation system and the Bokharo Thermal station. The project benefits the states of Bihar and West Bengal.

(*a*) *Tilaiya Dam* :—This is in Bihar; it is a concrete dam 1147 ft. long, having a storage capacity of 0·32 million acre feet. Height of the dam is 94 ft. Power proposed to be generated is 4000 Kws. This dam situated in the upper reaches of the Barakar river was inaugurated in Feb. 53. The dam has filled now for the fifth time in succession. The hydro-electric plant installed here is working very satisfactorily.

(*b*) *Konar Dam* :—Across the river Konar consists of a concrete dam 360 ft. in length and earthen dykes of a total length

of 12000 ft. The average height of the dam is 163 ft. above the bed level, and the capacity of the reservoir is 0·26 million acre ft. The power proposed to be developed is 40,000 kws. The dam work is completed and the water therefrom is being used for cooling purposes at the Bokharo Thermal station.

(c) *Maithon Dam* :—This is across the river Barakar in Bihar. This consists of a concrete dam 1300 ft. in length and earthen dykes of 10,473 ft. The average height of the dam is 158 ft. above bed, and the storage capacity 1·104 mil. acre ft. Power proposed to be generated is 40,000 kws. The dam work is almost completed and it was filled for the first time in June 56. Erection of the intake gates of the power-house & the power-house work is also completed.

(d) *Panchet Hill Dam* :—This is the biggest of the dams here ; length of the concrete dam is 1175 ft. and the earthen dykes 20,140 ft. Maximum height of dam is 135 ft. and the capacity is 1·22 mil. acre ft. Power proposed to be generated is 40,000 kws. This is arranged to be completed by Jan. 58 and the power-house by Oct. 58.

(e) *Durgapur Barrage* :—The four dams noted above will serve as reservoirs to feed the barrage down at Durgapur. This barrage 38 ft. high will divert the water of Damodar into an irrigation-cum-navigation canal 90 miles long connecting Hoogli on the left side and 121 miles of the main canal. The total length of the main canals and distributaries is 1533 miles. The barrage and the bridge have been completed and part of the canal also; the whole length is propsed to be completed by June 58.

One of the main objects of the scheme was to tame and discipline the notoriously turbulent river Damodar. The four dams together will ensure the valley lower down against floods (up to 1 mil. cusecs as against the highest recorded flood namely 6·5 laks cusecs).

The project supplies water to a perennial irrigation of 9·67 laks acres and generates 2·4 laks kws. It facilitates growth of industrial concerns, electric railways, and cheap transportation through its navigation canals.

3. Hirakud Dam Project:—Hirakud runs high among India's achievements since independence. It is the biggest irriga-

tion and power project completed so far in the country. The dam is across the river Mahanadi which is 533 miles in length and the catchment area is 51,000 sq. miles, above Naraj. This river is about 25 per cent larger than the Tennessee river (U. S. A.). During the dry season it has not got more than a few hundred cusecs of flow, but in the monsoons, the discharge is more than a million cusecs.

Even in 1937, Sir M. Visweswariah was the first to envision the problem that when a reservoir is constructed, it may prove useful in several other ways as well, for example, extending irrigation, generating electric power, etc. He said "once flood comes under effective control, the whole area may be transformed into a prosperous region." The flood detention reservoir was the only remedy for the river, as endorsed by Sir M. V. which was realistic in construction.

The foundation stone for the dam was laid on the 15th March 1946, by the then Governor, Sir Hawthorne Lewis, and it was inaugurated on the 13th of April 1948 by our Prime Minister Nehru who laid the first bag of concrete on the foundation.

Besides flood control, the stored water supplies perennial water for the first time to an area of 6·72 lakhs of acres in Sambalpur district. The stored water after generating hydro-power to the extent of 1·65 lakhs kilowatts at 75% load factor, will be utilised for the delta irrigation.

The reservoir level is kept at R. L. 590. A large number of deep sluices—a unique feature of this dam—has been provided, so that with this reservoir level as much as 7 lakhs cusecs can be passed down the river. Thus throughout the flood season, the reservoir will have maximum flood reserve capacity of 4·7 million acre feet available to absorb flood peaks.

Design:—The capacity of the Krishnarajasagara Dam in Mysore constructed in 1931 was 1 million acre ft. The capacity of Mettur Reservoir in Madras constructed in 1934 was 2·2 million acre ft. The third in 1954 was Thungabhadra, with 3 million acre ft. And now the Hirakud dam can hold 6·6 million acre ft. Its capacity may be said to be more than twice the combined capacity of all the 4 dams in the Damodar Valley Corporation.

For the design of spillways, various combinations of crest-gates, siphons and deep-set sluices were considered and comparative costs worked out, and from this, the most economical one was adopted. The result was 34 crest gates of 51′ × 20′ and 64 deep set gates of 12′ × 20·33′ in the lower part of the spillway were provided. The Hirakud dam is *unique in respect of the number of sluices.* One may wonder why so many sluices have been provided here. The Aswan Dam in Egypt has 180 sluices (40 sluices of 6·56′ × 8·2′ and 140 sluices of 6·56′ × 23′) and these act under a head of only 22 to 45 ft. and are of smaller size. The object of the sluices is to let out the silt from the reservoir as much as possible and increase the life of the reservoir. It is seen that though the Aswan Dam is more than 50 years old, there is no silting up. To operate the sluices a gate gallery of 2125 ft. in length is constructed in the body of the dam. This is the *biggest gallery ever constructed in the world.* Detailed calculations and model tests had to be done before the actual execution of this work.

The Hirakud Dam is distinguished for another feature. It is the *longest earthen dam in the world.* The flat topography of the country necessitated it. The main *earthen dam here is the highest in India,* being 190 ft. in depth over the river bed. A number of features such as heavy rock toe, cut-off trench, grouting, giant retaining walls are all carefully designed to ensure stability of the dam.

Hirakud Dam has the *largest Indian designed power house.* This is the first in which the design of a concrete power house for a large installed capacity is tackled in India.

In any modern design of a dam structure, gates formed an important unit, and in the case of this work it is specially important due to the number and the large size of the gates. All these were designed and built by Indian engineers only.

Dykes :—The main dam is flanked by dykes, the left side consisting of five bits, total length 6 miles, and the right side a single one of 7 miles in length. Thus including the main dam of masonry and earthwork, the total length of the dam with the dykes is 16 miles, and this is the *longest in the world.*

Other features which are also of exceptional dimensions and are interesting are the following.

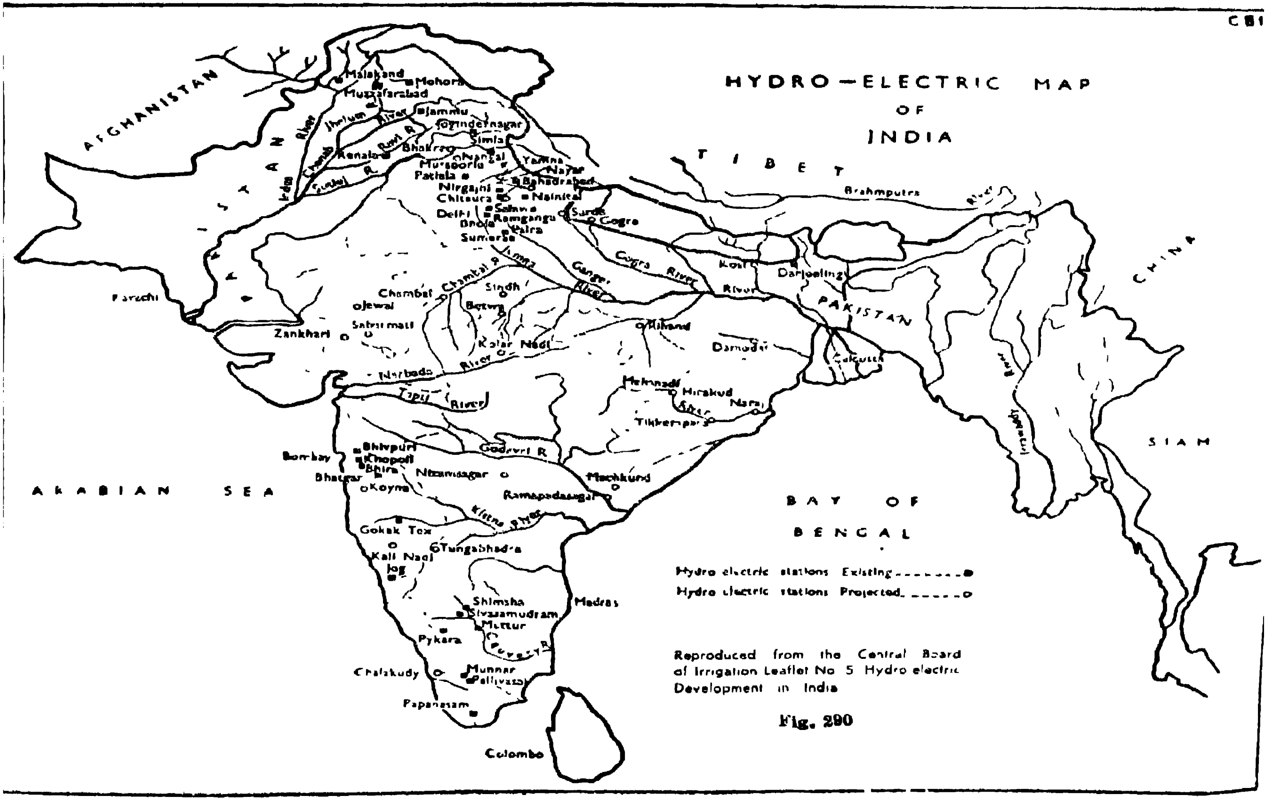

CBI
HYDRO-ELECTRIC MAP
OF
INDIA
AFGHANISTAN
TIBET
CHINA
SIAM
PAKISTAN
ARABIAN SEA
BAY OF BENGAL
Brahmputra River
Malakand
Mohora
Muzzafarabad
Jhelum River
Jammu
Jogindernagar
Simla
Ravi R.
Bhakra
Nangal
Renala
Chenab
Sutlej R.
Indus River
Mursoorlu
Patiala
Yamna
Nayar
Bahadrabad
Nirgajni
Chitaura
Nainital
Salawa
Delhi
Ramganga
Bhola
Vaira
Sumerse
Sarda
Cogra
Cogra River
Ganges River
Jumna
Kosi
River
Darjeeling
Chambal
Chambal R.
Sindh
Betwa
Jewal
Sabarmati
Zankhari
Karachi
Kolar Nadi
River
Rihand
Damodar
Calcutta
Narbada
Tapti River
Mahanadi
Hirakud
Narai
Tikkerpara
Bombay
Bhivpuri
Khopoli
Bhira
Bhatgar
Koyna
Nizamsagar
Godavari R.
Machkund
Ramapadasagar
Kistna River
Gokak Tox
Tungabhadra
Kali Nadi
Jog
Shimsha
Sivasamudram
Mettur
Madras
Pykara
Cauvery R.
Chalakudy
Munnar
Pallivasal
Papanasam
Colombo
Hydro electric stations Existing
Hydro electric stations Projected
Reproduced from the Central Board of Irrigation Leaflet No 5 Hydro electric Development in India

Fig. 290

In the 2091 ft. length of the spillway, there are 64 sluices at bed level with a size of 20·3′ × 12′ each, and the spillway is surmounted by radial gates of 51′ × 20′ and they are 34 in number. The spillway is a concrete structure, and in its belly, there is a gallery of size 11′ × 33′ for the full length. This is intended for the operation of sluice gates. The Hirakud spillway is perhaps *the largest in the world pierced with so many holes at the bottom* that " the two sections can pass down at maximum reservoir level a flood discharge of 1·5 million cusecs. "

Magnitude of the dam :—The total volume of concrete for the spillway and power house is 22,900,009 c. ft. and the total rubble stone in cement mortar in the power dam is 15,200,000 c. ft. A 25 ft. roadway on the main dam and the dykes provide a drive-way for 20 miles around the reservoir. The total volume of the earth used for this dam is more than the sum total of the earth used in the lower Bhavani, Gangapur, Maithan and Panchet Hill Dams.

The Hirakud Dam has already helped in moderating the floods in the Mahanadi Basin. It will help large tracts of lands in Sambalpur and Cuttack to grow bumper crops. More than a lakh of acres is already irrigated here. Industries, like the new steel plant at Rourkela and the alluminium factory at Sambalpur are fast taking shape. The power is also utilised for the paper factory at Brajrajnagar, the cement factory at Rajgangapur and the textile factory at Chowdwar. In addition to the above, power is utilised for rural electrification. The final development proposed is 2 laks of power and 2 million acres of irrigated land in both Kharif and Rabi seasons.

The Bargarh canal was opened and water was let out for irrigation on the 7th Septmber 1956. The canal is designed to carry a discharge of 4800 cusecs and commands a net cultivable area of 3·8 laks acres. In the two seasons Kharif and Rabi, over 5·25 laks of acres is receiving water from the reservoir.

On 13th January 1957, the dam being completed, was inaugurated by our prime Minister Sri. Nehru.

Hirakud dam is " a symbol of India's striving to gather the shattered forces and harness them to the service of the common man. Turbulent waters of the Mahanadi, which recently trod a path of recklessness, not unoften accompanied with devastation

of colossal magnitude, have today been tamed and harnessed by the hand of man to serve their previous victims. The expanse of water that meets one's eye as far as it can travel, is truly a reservoir of Orissa's future prosperity."

This project has been constructed exclusively by Indian engineers. "But it has not been well publicised; otherwise by this time, it should have been considered as the *pride of Indian Engineering achievements*".

Hirakud Project Details

Main Dam: Length-Concrete and Masonry 1640–2128 –3768 ft.

Earthen dam 11,980 ft.

Dykes: Left, in 5 parts–32,275 + Right 35,500 = 67,775 ft. Total length.

Masonry Dam: Maximum height 200 ft.

Earthen Dam: do 195 ft.

Under sluices: Number Left 40 + Right 24 Total 64
Size 12′ × 24′ each
Sill level 510.00 R L

Spillway: Crest level 610.00 R L
Number of crest bays
left 21 + right 13 + 34
Size of gate 51′ × 20′

Reservoir: Full Reservoir Level 630.00 R L
Capacity, Gross storage 6·60 Mil. acre ft.
At R L 590·00 Dead storage 1·88 Mil. acre ft.
Live storage 4·72 mil. acre ft.

Canal System: Irrigated area 6·72 laks acres
Main canal and branches-Length 91·5 miles
Distributaries and Minors 460 miles
Water courses 9500 miles

Delta: Irrigated area 18·67 laks acres
Main canals and branches 436 miles
Distributaries and minors 2015 miles

4. Nagarjuna Sagara Dam Project:—This is the second huge Multipurpose project of the National planning of the

Krishna River system, the first being the Tunga Bhadra Project at Mallapuram in Bellary District, Mysore.

The Krishna River has a heavy discharge of about a million cusecs. Now already about 25% of it is being utilised in the Krishna Delta and about 25% is arranged to be utilised under the Tunga Bhadra Project. This project-Nagarjuna Sagara-proposes to utilise another 25% of the river discharge.

The Krishna rises near Mahabaleshwar in the Western Ghats and after traversing portions in the Bombay and Andhra states, enters the Bay of Bengal, the total length being 775 miles. The flood discharge of the river is twice that of the Nile or Kaveri.

The reservoir is named after the famous Nagarjunacharya, founder of the Mahayana School of Budhism who lived in the Sri Parvata Hills near by, now called Nagarjuna Konda. This Nagarjuna Konda was the capital of a line of kings called Ikshwakus and the founder of this dynasty was one Santamala I. Recently the Archaeological Department of India began excavations in this place and relics of Budhism, site of an ancient city, coins, etc. were all discovered in the Mahachaitya. Traces of wharfs for anchoring boats are also seen here which shows that navigation existed up to this point. It is now proposed to construct a museum on the hilltop at Nagarjuna Konda and preserve all these relics, and develop the place as a tourist centre.

Lower down near Vijiavada, Sir Arthur Cotton constructed an anicut in (about) 1857, and 40 miles lower down, the river joins the Bay of Bengal. In its lower course, the river was called Mailsolos by Ptolomy and hence the name Masulipatam to the town near by.

The foundation of the dam is excellent, and all the required materials viz, stones, lime, gravel, etc. are all available near by.

The dam is situated in a beautiful gorge–fairly steep–and the dam site is only about 13 miles from the Railway station-Mancherla. The height of the dam is 387 ft. *the highest dam to be constructed in masonry.* It (the dam proper) is proposed to be built of rubble masonry with 1–4 of cement and sand mortar for the construction, where the compression stress is about 15 tons, and on the reservoir face to ensure impermeability, a proportion of 1 to 3 with puzzolana is recommended for the mortar. The advantages claimed here are

(1) Less consumption of cement which for the present is not available at a low cost.

(2) Less equipment requiring foreign exchange.

(3) Employment of most of the local labour.

(4) Savings in the overall cost by a good amount (about 4–5 crores). The reservoir is intended to store 10·7 Mil. acre ft. and this capacity is more than either Bhakra or Hirakud (Bhakra is 7·4 and Hirakud 6·6) or about 3 times the capacity of water that is now being used in the Krishna.

Two channels are proposed to be taken, one from the right bank for a length of 276 miles to irrigate 18·58 laks acres, and another from the left bank, for a length of 140 miles to irrigate 12·19 laks acres, and in addition to this, about 10·5 laks acres already under irrigation will be assured with water supply. Thus the total length of the channels will be about 426 miles to irrigate about 34 laks acres (more than 1½ times the present Godavery delta area) which will produce 12 laks tons food grains per year. The lands proposed to be irrigated are situated in the arid districts of Nalgonda and Khemmam of the old Hyderabad state and Godavari, Krishna and Nellore of the old Andhra state—Now all this is in the present Andhra Pradesh where the rainfall is scanty, precarious, erratic and flashy. *The total discharge in both the channels* put together is 21,000 cusecs, *the biggest in the world.*

The work is proposed to be completed in 2 stages or phases. In the first phase, it is proposed to provide water facilities for 9·7 laks acres under the right bank channel, 7·9 laks acres under the left bank and 1·5 laks in the delta area. The dam will be raised to 525·00 R. L. and the right bank channel with 303 thousand million c. ft., for a length of 140 miles and the left bank channel with 161 thousand mil. c. ft. for a length of 108 miles to irrigate 7·9 laks acres.

In the second stage, the dam is proposed to be raised to the full height R. L. 590·00 and water allowed for 122·02 laks acres and power to be developed to 8·71 laks kws.

After the construction of the dam, the submerged area is mostly all uninhabited gorge tract and as such will not affect the people. It is economical as the compensation to be paid is small. It is intended to benefit about 31·83 laks acres and the work is proposed

to be completed in about 10 years time. The inauguration ceremony was made by our Prime Minister Sri Jawahar Lal Nehru on the 10th December 55. The return on the project is said to be 4·23 % and including power it is 5·63 %.

The construction of the dam will store a large quantity of water and it will have a good effect on the climate of the locality. The project when completed will be one of the best beauty spots of the world and will attract a large number of travellers.

Details of the Project

Dam proper

	Masonry Dam Spillway length	1880 ft.
	do Non-spillway length	2020 ft.
	Composite Dam length	4060 ft.
	Height of masonry dam above foundation	387 ft.maximum
Reservoir :	F. R. L. of the reservoir	590·00
	Water spread at 590·00	110 Sq. miles
	Gross storage capacity	9·3 Mil. Acre ft.
	Live storage	5·39 Mil. Acre ft.
Channels :	Right Bank	Left Bank
	Length of canal 276 miles	140 miles
	Sill of offtake channel 490·00	490·00 R. L.
	Full supply discharge at head 21,000 cusecs	11,000 cusecs
	Section of maximum discharge	
	Bed width 155 ft.	134 ft.
	Depth 20 ft.	15 ft.
	Quantity of water utilised 303 T. Mil c.ft.	101 T. M. c.ft.
	Area proposed to be irrigated 18·58 laks acres	12·19 laks acres
	Cost 61.10 crores	26·20 crores
	Power proposed to be developed	75,000 Kws.
	Total cost of the project	122 crores

5. Tungabhadra Project comprises constructing

(a) A dam across Tungabhadra river 7942 feet long. This has almost been completed excepting final erection of Gates, and

(b) Two canals on either banks of the river.

The left bank canal is 127 miles long and the right bank canal is 217 miles.

Left Bank Canal: The left bank canal serves Raichur District to irrigate 5,80,000 acres. Canal up to mile No. 47 is completed and on the remaining length, it is in progress. During the First Five Year Plan, the dam and the canal up to 32 miles were completed and an irrigation of 16,316 acres had also been done to the end of March 1956. Out of the total plan provision of Rs. 1609 lakhs for the left bank project, the expenditure incurred to end of March 1956 was Rs. 1305 lakhs. The Project is now continued in the Second Five Year Plan with a spill-over of 630 lakhs and it is expected to be completed in 1959 A. D.

The full development of the proposed anicut of 5,80,000 acres is proposed to be completed by 1967 A. D.

Right Bank Canal: Out of the total length of 217 miles, 106 miles length of canal is in the Mysore limits, to irrigate an area of 92,339 acres, out of the total of 2,49,258 acres proposed under the whole canal. Up to the end of the First Five Year Plan, 25000 acres have been irrigated and the remaining anicut of 67,339 acres is expected to be irrigated by the end of the 2nd Five Year Plan period.

IInd Five Year Plan:

Tungabhadra High Level Canal:—This scheme provides for the tentative utilisation of 50,000 M. c. ft. of water from the Tungabhadra Reservoir through the High Level Canal, with a maximum supply of 4,000 cusecs to be drawn from the reservoir.

This is a joint venture of Mysore and Andhra Prdaesh Governments as the canal serves the areas in the Bellary District of the Mysore state and the Ananthapur and Cuddapah Districts of Andhra-Pradesh. The scheme was scrutinised by the Planning Commission and included in the IInd Five Year Plan. In the inter-state conference of June 1956 A. D., it was agreed that the waters would be shared by Mysore and Andhra in the ratio 35:65. The work of the main canal would therefore be under the Tungabhadra Board, and distributaries will be done by the respective governments. The area proposed to be irrigated in Mysore is about 1,50,000 acres.

The first 25 miles of the main canal will be run in very difficult country involving long and deep cutting, side walling and finally a tunnel between the 25th and 26th miles. After crossing Hospet-Bellary-Raidurg Railway lines, the canal crosses Chikka-Hagri and Pedda-Hagri rivers by means of regulators. The canal will be lined throughout to minimise seepage losses. The probable cost of this scheme is 23·35 crores.

Rajulibonda Diversion Scheme (Raichur District):—This is a diversion scheme consisting of constructing an anicut 2690 ft. long across the Tungabhadra river, 80 miles downstream of the Tungabhadra project with 72 miles of canal on the left bank. The canal in the Mysore limits is 27 miles and the remaining length is in the Andhra Pradesh. The irrigation proposed under the entire scheme is 1,15,000 acres of which 74,000 will be in Mysore limits. This scheme was taken up by the former Government of Hyderabad for the utilisation of 850 cusecs of Tungabhadra waters alloted to that state as an offset against Cuddapah-Kurnool Canal on the Andhra side. The cost of this scheme is 433 lakhs. During the First Five Year Plan, a provision of 171 lakhs was made for this scheme of which an expenditure of Rs. 116 lakhs has been booked to the end of March 1956.

Anicut on the left side, with head sluice and scouring sluices has been completed. On the remaining length of the anicut in Andhra limits, the foundations have been filled up, but the anicut could not be raised to full *crest* level for want of settlement of the basic issues like submergence. On account of this no irrigation was possible even though the canal in the Mysore limits has almost been completed. This scheme is continued in the Second Five Year Plan with a spill-over amount of Rs. 150 lakhs for the whole scheme and is expected to be completed by 1959 A. D., if the flood bank of the Andhra side is constructed.

7. **Ghataprabha Left Bank Canal** :—This scheme is a part of the main scheme known as 'Ghataprabha Valley Development Scheme'. It comprises constructing the left bank canal (44 miles) with its branches (33 miles) taking off from the existing weir at Dhupdal across Ghataprabha river. The capacity of the canal at its head is 1500 cusecs which is to be raised to 2000 cusecs ultimately. The canal with its branches is expected to

irrigate 1,24,000 acres in this reach. The cost of this scheme is Rs. 545 lakhs. This scheme was included in the First Five Year Plan and the work is almost completed according to plan. The actual area irrigated at the end of the plan period is 13,325 acres, even though irrigation facilities were provided to cover an area of 76,000 acres. This scheme covers the scarcity parts of Belgaum and Bijapur Districts.

Ghataprabha (Second Phase) : Under the First Five Year Plan, the left Bank Canal, mile 0 to 44, has been taken up and completed which is known as First Phase of the Ghataprabha Valley Development Scheme. During the Second Five Year Plan, the second phase of the main scheme will comprise

(a) the construction of Hadalga Storage reservoir across Ghataprabha river and

(b) the Extension of left bank canal, Mile 45 to 73, proposed to be taken up now with its branches.

This part of the scheme is estimated to cost of 999 lakhs to irrigate an additional area of 1,30,000 acres.

(a) *Hadalga Dam* :—Across river Ghataprabha near Hadalga village of Kolhapur District (Bombay State) a storage reservoir to impound 32,000 M. c.ft. of water is envisaged by constructing a masonry gravity dam 1600 ft. long and 130 ft. high above the deepest river bed.

The storage dam is intended to feed the left bank canal during non-monsoon months to augment supplies for an irrigation of 2,54,000 acres under the left bank canal. The cost of this dam is Rs. 498 lakhs. The proposals of this storage dam are under scrutiny of the Central Water and Power Commission, and work will be started soon after the approval is received and the land made available by the Bombay State.

(b) *Extension of the Left Bank Canal* (Miles 45 to 73,):—The canal in this reach will have a carrying capacity of 830 cusecs at its head to irrigate an area of 1,30,000 acres under it. In this reach the length of the main canal is 29 miles with 75 miles of branch canals. The total command under this canal is 1,87,900 acres. The cost of this reach of the canal is Rs. 501 lakhs. Work on the main canal in miles 45 to 51 is in progress.

Both the storage dam at Hadalga and the extensions of the left bank canal are expected to be completed by 1961 A. D,

8. Malaprabha Irrigation Scheme (Belgaum District) '— To harness the waters of Malaprabha river, a Major tributary of the Krishna, an irrigation scheme, comprising

(a) a storage dam at Bhutewadi,

(b) a Pick-up weir at Manoli 50 miles downstream of the storage dam, and

(c) an Irrigation system on the right bank consisting of 91 miles of Main canal and 25 miles of branch channels,

to command an area of 3·3 lakhs acres in the scarcity areas of Navalgund, Nargund and Ron Taluqs of Dharwar District and Saundatti Taluq of Belgaum District has been contemplated by the former Government of Bombay.

The actual area served by this scheme is 1,50,000 acres. The approximate cost of this Irrigation Scheme is Rs. 13 crores.

The storage dam at Bhuttiwadi will be 2510 feet long and 164 ft. high to impound 11,000 M. c. ft. of live storage. The water so stored will be let down in the river during non-monsoon months and diverted for irrigation purposes by a pickup weir constructed 50 miles downstream of the river near Manoli, The pickup weir will be 318 feet long between abutments and 60 ft. in height. From the right flank of this pickup weir the right bank main canal will be taken off with a head capacity of 1700 cuse s. The canal will have nearly 100 masonry works and a tunnel of nearly 4 miles long, from mile No. 1 to 5. The alignment on the Main canal for nearly 50 miles have been completed and in the remaining length it is in progress.

During the Second Five Year Plan, a sum of Rs. 850/- lakhs is proposed to be spent, leaving the balance to the third five year plan. Further investigations reveal that a single dam at Manoli would be feasible; the additional advantages being, increase in the area under irrigation from 1·5 lakhs to 2·3 lakhs acres, and decrease in the cost of the scheme by about Rs. 200 lakhs. The final stages of investigations, as also the preparation of estimate, report and other details are nearing completion.

9. Dharma Storage Reservoir:—(Dharwar District)- Dharma river is the branch of Vardha river, a tributary of

Tungabhadra. The waters of the Dharma river are proposed to be harnessed near Yamangalli by constructing an earthen dam, 4750 ft. long and 66 ft. high to impound 784 M. c.ft. of water for providing irrigation facilities to the existing ayacut of 9000 acres, with a further irrigation of 5000 acres in Dharwar District. The whole of the irrigation is under the Right Bank Canal, 17 miles long with 18 miles of branch channels. This scheme has been investigated in detail and accepted by the Central Water and Power Commission and is at present with the Planning Commission. The cost of this scheme is Rs. 70 lakhs. During the Second Five Year Plan, it is proposed to be taken up and completed by about 1960 A. D.

10. Malampuzha Dam:—The first Irrigation Project of Malabar, the Malampuzha Project, for which Malabar was agitating for the last forty years, has been completed (the Dam proper)

The Chief Minister of Madras unveiled a tablet on 10th October 1955, commemorating the completion of the dam.

The project was first considered in 1914 and a preliminary report was prepared in 1924, with an estimate of Rs. 60 laks. For some reason or other, the project was again taken as a bigger scheme in 1946, and the investigation was completed in 1947 and the scheme was sanctioned in 1949 and it was estimated at Rs. 380 lakhs.

The Malampuzha project consists of (a) a dam across the Malampuzha river, which is a tributary of the Bharathapuzha and located at a place five miles away from Olavakkot and (b) a system of canals and channels.

The total length of the dam is 6066 ft. of which only 729 feet is of earthen construction and the remaining portion, of masonry. The dam is in five sections. The maximum height of the dam above the deepest portion is only 125 ft.

The reservoir capacity is 8,000 million c.ft. At the site of the dam, the catchment area of the river is 57 sq. miles. The water-spread area of the reservoir is 9·6 sq. miles. This consists of 2648 acres of wet land, 191 acres of garden lands, 300 acres of river Poromboke, 76 acres of unassessed waste land and 31·06 acres of dry land. The reservoir will submerge no village or hamlet.

The estimated compensation for acquiring the submerged area is Rs. 40 lakhs. The project is estimated to irrigate 40,000 acres.

On the left flank of the main arm of the dam, the main irrigation sluice is located with three vents of $5' \times 6'$. For power generation (at future use), two penstock pipes are located adjacent to the sluice. Also, another sluice is provided on the right flank, with a single vent, to irrigate some lands. Pipes are laid across the dam, adjoining this sluice, for water supply to the Palghat Municipality.

The spillway over the dam is located near the bed. It is provided with four gates of $36' \times 15'$.

The distributary system consists of a 20 mile long main canal and minor channels totalling over a hundred miles in length. The canal will take off from the left flank of the main dam, with a full supply discharge of 750 cusecs, and with a bed width of 60 ft.

The project was inaugurated on 27th March 1949 and the preliminary works were completed in a year.

On a hill on the left flank of the dam, a special building, 'Lake View,' is erected, from where the dam gives a picturesque view, and added to the scenery are the long ranges of mountains, with waterfalls, in the background. Planned gardens are laid to beautify the front view of the dam. There are possibilities of making the area a tourist centre.

11. **Mayurakshi Dam West Bengal**:—This is the first of the River Valley Projects completed. It was completed (excepting the Power Plant) six months ahead of the schedule in 1954.

It consists of a dam at Massanjor and 4 Barrages at Tilpara, Kopal, Dwarka and Brahmani and a weir at Bakreswar. It is intended to irrigate 6 lakhs acres in Bihar and produce 4000 K. W. of power.

It is seen that it has increased the yield at 6 mds. of paddy per acre, and as such an additional income of 3 crores per year to the ryots. It is proposed to change the crop pattern and if this is successful, the additional income will rise to 8 crores.

12. Bhavanisagara Dam :—This project was thought of as early as even 1874, and due to various reasons especially that it is not paying, as usual with the British rule, it was kept back. Now after the achievement of independence the project was pressed and was got sanctioned in 1947.

The project consists of constructing a dam, and taking a canal from it to irrigate the arid tracts of Coimbatore and Tiruchi districts and to give protection to old lands.

The work was started (preliminaries) early in 48, but the actual work in October of the year, and has been now completed.

Salient Features of the project. Dam Masonry portion Length 1523 ft.

Earthen Dam-Length-left 15250′ and right 12069′ = Total length about 5½ miles.

Canal Sluices — 5 of 6′ × 10′ to irrigate about 2·07 lakhs acres. Flood disposal—An ogee type spillway 396 ft. long containing 9 spans of 36 ft. each with 8 intervening piers

River sluices—9 of 6′ × 10′

Masonry Dam. This portion consists of 6 different sections from left to right, they are

357 ft. of Non-overflow portion

395 ft. of overflow or spillway portion

200 ft. of river-sluice section (9 vents of 6′ × 10′)

140 ft. Penstock section (4 of 8 ft. diameter)

137 ft. Canal-sluice section (5 of 6′ × 10′)

Non-overflow section 293 ft.

The dam is designed as a gravity dam, with all modern improvements. It is 204 ft. above the deepest foundation, and 140 ft. above the river bed. Due to bad foundation at higher levels—talc schists—deeper foundation more than 40 ft. below sanctioned had to be taken. To reduce uplift the foundation has been well grouted and a drainage gallery of 5′ × 7½′ is also provided. Three transverse galleries are also provided, to give access to the main drainage gallery, and also to lead the seepage water. The masonry dam was begun in Oct. 49, and completed

in June 52. Earthen Dam—This is of the Zone type, consisting of 7 parts as per the latest practice in earthen dam design. There is a central impervious zone enveloped by semi-pervious zones. To minimise seepage, and to have proper bond between the foundation and the earthwork, a cutoff trench with cutoff walls, and a grout curtain below each of the cutoff walls is provided. At the rear portion, a filter with graded materials is also provided. The consolidation has been done with the optimum moisture content.

The earthwork began in April 49, and the sluices were opened in September 52.

Canals. The length of the canal is 124 miles. The head sluice has 5 openings of 6′ × 10′ capable of discharging 2300 cusecs for irrigating 2·07 lakhs acres. The bed width at beginning is 111 ft. and depth 8·6 ft.

Spillway. The spillway consists of an ogee-weir of length 396 ft. This discharges about 1,22,000 cusecs, and together with the river-sluices, a total of 1,56,700 cusecs. To dissipate the energy of the falling water from a height of 113 ft., a stilling basin with baffle blocks is provided. This arrangement was adopted after research work in Poondi Research station.

Instead of the usual free-roller type of gate, a fixed-roller type gate is tried here. The hydraulic pressure on the gate is taken up by a skin plate supported by 5 lattice trusses of mild steel. The trusses transmit the load to the 2 end beams, each of which is equipped with cast steel rollers fitted with roller bearings, oil seals, etc. The rollers travel on tracks of the rocking type. The leakage of water is minimised by staunching pipes at the sides and rubber belts at the bottom of the gate. The gates are operated by independent electrically-driven hoists, at a speed of about 1 ft. per minute. Facilities for manual operation is also provided, in case of power failure.

The whole equipment was designed, manufactured and erected by the departmental authorities, at a cost of 9 lakhs of rupees, which is comparatively economical.

The opening ceremony of the shutters was done by Shri. Kamaraj Nadar, Chief Minister of the state on 19th Aug. 1952.

Second 5 year Plan-Irrigation Works-Mysore.

(a) Minor Irrigation: The plan contemplates an outlay of Rs. 657 lakhs on minor irrigation works including restoration of minor tanks and for desilting of tanks. Though the new area brought under irrigation (12,000 acres) by these schemes is comparatively small, irrigation of the existing lands (2,25,000 acres) will be improved, and the schemes will provide work for unskilled labourers during off season.

(b) Major Irrigation Projects:—The main projects are:

(1) The Kabbini reservoir across the Kabbini in the Mysore district to impound 12,000 million cubic feet. This will be used for irrigating 6,000 acres and for augmenting the flow of the river during summer to keep up the steady generation of power at Shivasamudram and Shimsha. After this reservoir is constructed, the storage in K. R. S. reservoir now reserved for power purposes will become available for irrigation and a further extent of over 12,000 acres will be brought under the K. R. S. reservoir irrigation.

(2) The reservoir across the Hemavati near Gorur. This will irrigate 45,000 acres of land, and in addition service a balancing reservoir for storing water from the river Hemavati in good years, after meeting the demands of established irrigation lower down the river.

The remaining projects are across smaller streams and will benefit areas, where irrigation facilities are wanting. The total cost of major irrigation schemes included in the Plan is Rs. 2,27,189, lakhs including a spill over of Rs. 1,152 lakhs from the first plan.

In the second five year plan, it is heard that Mysore gets only 85 crores.

HYDRO-ELECTRIC SCHEMES

1. Power can be obtained from (1) exhaustible sources like coal, mineral oil, peat, natural gases, etc. and (2) inexhaustible sources like waterfalls, winds and tides. The production of power from sources like tides and winds is limited. In countries where coal is scarce, such as India, Switzerland, Norway and Sweden, large hydro-electric plants have been developed.

Under certain favourable conditions, river waters provide a cheap source of power, either directly or through the generation of hydro-electric power. When water is stored for irrigation, it is generally possible to utilise the water for generation of power also. Power is a function of the "head" and "flow". Generally speaking, the power which can be generated at a site, may be approximately estimated by the formula,

$$\text{Power} = \frac{\text{Head in feet} \times \text{Flow in cusecs}}{12}$$

Cheap electricity is essential for the development of a country, and the extent of its use is an index of the standard of living and national development of a country. Cheap power, as the vital force behind industrial development, is one of the important solutions of the economic problems.

Water-power satisfies our fundamental need of cheap power. Water power exploitation developed *pari passu* with colonisation of lands, and the use of water wheels can be traced back to the middle ages. Both from the point of view of permanency, as well as installation and operational economy, it is best to develop power from water. Every H. P. of hydel power is estimated to save Rs. 40 worth of coal per annum.

2. The main requirements of water power generation are :— (1) Availability of water in sufficient quantity throughout the year.

(2) Sufficient head of water.

The cost of power generation and the demand for the generated power should also be taken into account when planning a hydel power project.

The water for the generation of power may be available naturally, or it may be made available artificially. The natural perennial flow of a river may be utilised for power generation, or when the flow in the river is seasonal, water may be made available from canals taken from storage reservoirs.

The topography of the country determines the head at which the water is available. The head may be due to a natural fall, or it may be provided artificially.

3. Hydro-electric Installations ;—Every hydro-electric installation consists principally of the following :—

(1) Work (usually dams or weirs) for controlling the water level in the upstream reservoir, and thus creating the necessary head.

(2) Diversion works to control and divert the water-supply from this reservoir to the intake channel (or head race) leading to the power house.

(3) Intake channel or penstock to lead the water to the hydel power house.

(4) Power house usually at the foot of the fall. This contains hydraulic turbines, electric generator together with switch gear and controlling apparatus. Here the kinetic energy of the falling water is converted into mechanical energy by the water turbines, and this mechanical energy works the electric generators which produce the electric energy.

(5) Tail race leading the water used in the hydraulic turbines back into the river.

Fig. 289 gives a general idea of the component parts of a

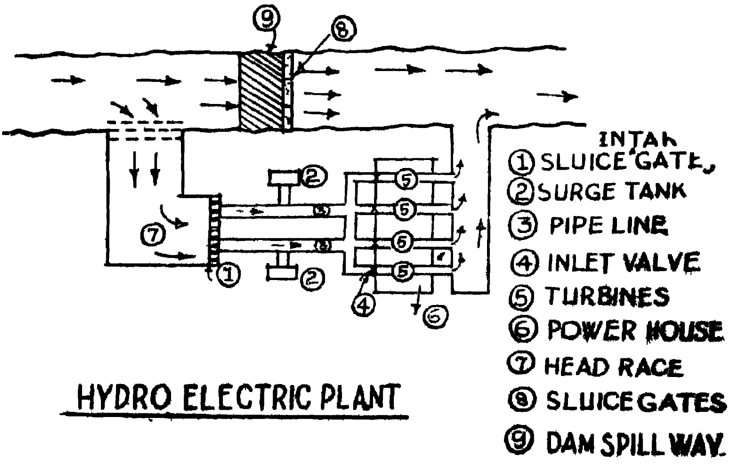


Fig. 289.

hydro-electric scheme. A dam is constructed across a river to raise its height, and the surplus water spills over the head of the

dam or spillway, thereby taking the flood water to the lower level. Sluice gates are also provided at the dam to assist this purpose, and also to provide a means of uncovering the bottom of the intake sluice gates for inspection and overhaul.

The intake is a small pool or canal and is for the purposes of linking the river with the pipe lines. At the river end of the canal is a metal screen or strainer which prevents solid matter and rubbish from entering the pipe lines, and at the other end are the sluice gates which are for the purpose of isolating the pipe lines from the water supply. The closing of the sluice gates enables either of the pipe lines to be drained, thereby giving access to the pipes, surge tanks and inlet valves for inspection and repair. The pipe lines are generally of reinforced concrete for low heads when the head does not greatly exceed 100 ft. but riveted steel pipes are used for high head schemes Automatic valves are also installed on all pipes, and these operate to shut off the water if a pipe bursts. The walls of the pipe lines are thickened as the height increases so as to withstand the increasing pressure.

Near the power house, the pipe lines are resolved into distributors, one for each turbine. The water after passing through the turbine runner, makes its way back to the low level part of the river. The scheme is essentially the same in outline, either for high or for low heads.

4. Water-power available :—The power available at the intake valve of the turbine can be easily calculated. If Q is the amount of water passing through the pipe line in cusecs, and H is the head or the height in ft. through which the water falls, then the weight of water passing through the pipe line in one second is 62·4 Q lbs. (since 1 c. ft. of water weighs 62·4 lbs.) The power is 62·4 QH ft. lbs. per second. Since there are 550 ft. lbs. per second in one horse-power, the theoretical horse power of the falling water is,

$$\frac{62{\cdot}4\ QH}{550}$$

Thus for the same power in both cases, a high head installation will be characterised by a small volume of water per second, and a low head installation by a large volume of water per second, and so the same type of turbine will not be suitable for both schemes.

The actual useful power obtainable depends upon the efficiency of the various parts of an installation. Assuming an overall efficiency of 75 per cent, the power available is

$$= \frac{62 \cdot 4\, QH}{550} \times \frac{75}{100} = \frac{QH}{12} \text{ H. P. (approx.)}$$

$$= \frac{QH}{16} \text{ kW (approx.)}$$

5. Classification of Schemes:—It is usual to classify hydro-electric power installations, depending upon the topography of the country, roughly under three headings.

(1) High head schemes utilising heads above 500 ft.

(2) Low head schemes where the head varies from 100 ft. down to a few ft.

(3) Medium head schemes from 100 to 500 ft.

(1) **High Head Installation**:—The source of power for a high head scheme is generally a waterfall, which provides a natural dam and spillway. When there is a great natural fall of the country along the course of the river, and the river flow fluctuates according to seasons, water is stored at the upstream end of the fall, by constructing a dam, and from this reservoir, water is taken in a pressure channel along the margin of the river to a valve house at the head of the pipe line. At this valve house, the flow of water in the penstock leading to the generating house is controlled. The generating station is situated very near the main river into which the tail race from the generating station discharges. For high heads, the Pelton wheel, or impulse turbine is employed.

(2) **Low Head Installation**:—Here a barrage or diversion weir is constructed across the river to produce afflux on its upstream side. The power house is built in half of the barrage spans. Gates to control the flood flow are fitted in the other half of the barrage spans. For low heads, the Francis, or reaction turbines are employed.

If the natural banks of the river are too low and cannot hold the raised water on the upstream side of the barrage, the intake (head race) and tail race channels are constructed for a greater length, having flatter slopes than that of the river. The power house is built across this open channel.

Instead of one installation, two or more may be conveniently arranged in series, the water from the uppermost turbines being successively led to the lower ones, which may be at some distance downstream.

(3) **Medium Head Installation** :—When there is a local fall in the bed of the river, an open diversion channel is constructed at the upstream end of the fall and the water led to the power house. For medium heads also, Francis type turbines are used. From the turbines the water discharges into the tail race as usual.

In some installations, the power house is located in the river bed itself near the dam and forms part of it. The water is delivered to the turbines without passing through the head race or penstock.

6. Hydro-electric Power Development in India :—India has immense water power potential, well distributed over the entire country and such areas, where the power potential is low or absent, are within economic transmission distance of potential water power sites.

Hydro-electric power development is 57 years old in our country. The first hydro-electric power station was opened in 1897 in Darjeeling. In India, the development of electric power has been comparatively slow, even though there are abundant potential resources for such development.

Until atomic power and solar energy come into the field, the development of power resources in India can only be from water, coal and oil. In India, where there is very little oil and the coal deposits—particularly high quality coal—are limited and confined to certain parts of the country, hydro-electric power plays a big part in development.

Year	Installed kW of public undertakings
1900	1,130
1920	133,847
1925	327,152
1948	1,399,410
1951	2,000,400

The capacity in 1954 and 1959 is expected to increase to [illegible] and 4,849,000 kilowatts respectively.

India compares poorly with some of the foreign countries in respect of per capita consumption of electric power.

A comparison of the relative power development in different countries in relation to their population is given below :—

Name of country	Population in million	Potential power in million kW	Present installed capacity in million kW	Installed capacity as a % of potential power	Consumption per capita in kW
Canada	10	38·0	7·7	20	4,000
Norway	3	4·5	2·4	53	3,090
Sweden	7	7·5	2·6	33	2,100
U. S. A.	133	45·0	8·5	34	1,660
U. K.	44	1·5	0·5	31	938
France	2	9·0	3·7	42	431
U. S. S. R.	170	100·0	22·4	22	...
India	330	40·0	2·35	6	

India's resources of hydro-electric power are potentially large and are estimated at 40 million kW. The total generating capacity in the country in 1950 was approximately 2·3 millon kW of which 1·7 million came from thermal stations and about 0·66 million kW from hydro-electric plants.

In India, the use of electricity is very limited, the average per capita consumption being only 14 kW per year. A notable feature in India is that the two cities, Calcutta and Bombay alone, (with a total population of only one-hundredth of the total population of the country), consume 30 per cent of the total generated electric power.

The power projects in the country, that are either under construction or under investigation, are estimated to create an additional power-generating capacity of about 7 million kW. When the major hydel projects under construction are completed, the hydel capacity in India is expected to increase from 0·5 million kW to 14 million kW.

The region-wise expansion in power generation anticipated is given below :—

Additions anticipated by 1955-56 in thousand kW

Projects and Places	Installed capacity.	Firm power.	Anticipated load.
Multipurpose projects:			
(i) Bhakra-Nangal ...	96	72	69
(ii) Damodar Valley ...	194	144	132
(iii) Hirakud Dam ...	48	24	24
Madras, Mysore, Hyderabad, and Travancore-Cochin ...	402	511	639
Bombay area ...	83	358	417
Bihar, Bengal and Madhya Pradesh ..	88	65	101
Uttar Pradesh ...	109	157	146
Projects in other areas ...	62	66	66
Total	1082	1397	1594

The increase in generating capacity due to the three multipurpose projects as noted above is seen to be 338,000 kW. This represents only the first phase of development, and it is estimated their generating capacity will increase to one million kilowatts.

Industries are the largest users of power, consuming about two-thirds of the total: domestic and commercial users come next. Irrigation takes at present only about 4% of the power consumed.

The amount of electricity consumed for different purposes in 1950 is indicated below :—

Different purposes	Consumption in million kW.
Industrial	2604
Domestic light and power ...	525
Commercial light and power ...	309
Irrigation	162
Other purposes	558
Total ...	4158

The total annual supply of water in Indian rivers is estimated at 23 million cusecs of which less than 6 per cent is utilised. The rest runs to waste. The hydro-electric resources in India are estimated, as already said, at 40 million kW. Thus there is wide scope for development and fortunately the State Governments and the Central Government have launched a number of multipurpose projects and schemes. (See Fig. 290).

7. Projects.

(A) Machkund Project:—This is the first Hydel station in Andhra. Power from the first generating unit of Machkund Hydro-electric scheme was switched on, on the 19th August 55, by President Sri. Rajendra Prasad. This marks the commencement of hydro-electric generation in the State.

"The Machkund power house is situated in the Koraput district of Orissa State, 120 miles by road from Vishakapatam via. Ananthagiri Ghats, Araku Valley in Andhra, and Padwa in Orissa. It is also accessible by road from Bobbili via. Salur in Andhra and Koraput and Jeopore in Orissa, a distance of again 120 miles.

"The Duduma Falls with a drop of 550 feet on the Machkund river are harnessed. The river is one of the sub-tributaries of the Godavary. In the composite Madras State, prior to the partition of Orissa, surveys of hydro-electric potentialities were carried out in 1929 and a comprehensive report drawn up in 1931 by Sir Henry Howard. With the formation of the Orissa State in 1936, Machkund river became the boundary for Madras and Orissa near the project site. The Machkund Hydro-electric project is a joint venture of Orissa and Andhra. Although jointly owned, it is being constructed under the direction and supervision of the Andhra Government Electrical Department.

"The ultimate power potential is 1,02,000 K. W. Half this capacity, viz., 51,000 K. W. is installed in the 1st stage. The power benefit and capital cost are shared by Andhra and Orissa in the ratio 70:30.

"The target date of September 1956 for the commissioning of the first generating unit envisaged in the composite state of Madras, had, after the formation of the Andhra State, to be advanced and construction schedules accelerated owing to lack of

Hydro-power in the new-born state. By working round the clock, new records were achieved. From foundations of the power house to completion of the first unit in eighteen months is a gratifying progress.

"The Andhra share of capital cost works out to about Rs. 15 crores initially, rising to about Rs. 19½ crores at the end of ten years."

"Headworks of the Machkund consisting of the generating station, dam and other civil works are also works common to Andhra and Orissa. The cost and power benefit is shared in the ratio 70 : 30, but the executive agency is entirely Andhra.

"Nandikonda Electric Project relates to Andhra as already mentioned, for general co-ordination.* The Nandikonda Control Board which is constituted for the execution of the Nandikonda Irrigation Project is likely to have general control over the hydro-electric project also.

"It is only the Sileru Hydro-Electric Project which is free from links with irrigation and inter-state joint control. It is therefore given priority over the Nandikonda Hydro-Electric Project."

"The Machkund Hydro-Electric Scheme; 2nd Stage, costing about Rs. 4½ crores and giving 51,000 K. W. of power is the No. 1 priority scheme in the Second Five Year Plan".

"The Tungabhadra Hydro-Electric Scheme, 2nd Stage, costing Rs. 4 crores and giving 36,000 K. W. hydro-power coupled with a thermal station at Nellore costing about Rs. 3½ crores and giving about 30,000 K. W. of thermal power to firm the seasonal power in the Tungabhadra Hydro-Electric Project is the 2nd priority scheme in the plan."

"Rural Electrification schemes costing about Rs. 11 crores and providing for supply to 1800 villages as mentioned already is the 3rd scheme giving priority 1 and 2 on a par with the Machkund and Tungabhadra schemes".

(B) Sileru Project. "The Sileru project site is 50 miles below the Machkund Project site on the Sileru river where it is the border between Andhra and Orissa. It utilises the tail waters

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* [illegible] is now called [illegible] Reservoir Project.

of the Machkund Project, and is essential for Andhra, since it is a pure power project not connected with irrigation, and also since it is to be the exclusive project of the Andhra State. In the Machkund agreement between Andhra and Orissa, the development of Sileru Hydro-Electric Project is mentioned as the exclusive right of Andhra.

"Nandikonda Hydro-Electric Scheme, 1st stage, with an expenditure of about Rs. 3¾ crores for the 2nd plan period and giving about 75,000 K. W. of power in the third plan period is the fourth priority scheme. Generation in this scheme will commence much ahead of the commencement of the irrigation supply, since generation is possible even when the dam is constructed to three-fourth of its full height.

"As already mentioned, the Sileru and Nandikonda projects will be taken up in the beginning of the second five year plan period, and completed in the Third Plan period, so that power shortage in the Third Plan period is avoided, and continued power development and, through it, industrial development is maintained.

"The headworks of the Tungabhadra Hydro-Electric Scheme consist mainly of the generating stations and common works to Mysore and Andhra. They share the cost and power benefits in the ratio of 80 : 20. The common works are under the control of a Board appointed by the venture."

(C) Honnemaradu Project. "The Rs. 40 crore giant Honnemaradu Hydro-electric Project is certain to be included in the Second Five Year Plan."

"Unless this project was implemented, there would not be any industrial development on an appreciable scale in the south, because the present power supply was unable to meet the growing demand."

Honnemaradu, officially called the Sharavathi Valley Hydro-electric Project, would be taken up in two stages. The first stage costing Rs. 22·87 crores to be included in the Second Plan, while the second stage would be taken up in the Third Plan.

The project will supply power at a low cost of about Rs. 700 per k. w. and the total quantity of power available, on completion of the project, will nearly be 7,00,000 k. w. The implementation

of the first stage of the scheme during the Second Plan would supply an additional quantity of 2,80,000 k. w.

Other important hydro-electric projects to be included in the Second Plan in Mysore, are

(a) Shimsha, Rs. 3·45 crores.

(b) Tungabhadra, Rs. 123·84 lakhs (producing 28,000 k. w. power of which Mysore's share will be 20 per cent).

(c) Bhadra Project, Rs. 82 lakhs.

Note: The target allotted for the Second Five Year Plan is about Rs. 150 crores.

Only 13 crores is given for the Honnemaradu Project in the Second Five Year Plan.

INLAND NAVIGATION

1. Introduction:—Communication or transport of materials through canals, rivers or other waterways, is called inland navigation.

One of the important uses of rivers is navigation. Even in the old days, the main arteries of communication were the main rivers of Northern India. This river traffic gradually declined with the construction of railways, but it can be seen even now in Assam, Bengal and Bihar, where there is more water transportation. The other cause for the decline is that irrigation has drawn the bulk of dry weather flow of the rivers. Inland transportation had never been large in Southern and Central India, because here the water in the rivers is not sufficient for navigation (and only small boats are used). The rivers Godavari, Krishna and some others can be navigated at present also, at their tail ends.

Water Transport:—The total mileage of navigable inland waterways in India is about 5760 miles. Of these only about 3000 miles are at present navigable rivers and the rest are canals and backwaters. Of the 3000 miles. Of navigable rivers, more than half are in the Ganga and the Brahmaputra and this length is mostly navigable by steamers.

Due to the neglect of water transport after the advent of railways, the waterways in India have deteriorated considerably. There are some coastal canals fed by salt water along the East

and West Coast. For ex. the Hijilli Tidal Canal in West Bengal. The Orissa Coast Canal, the Buckingham Canal in Andhra Pradesh and Madras, and the canals and backwaters in Mysore and Kerala.

A master plan, as noted below, for inland navigation has been drafted by the Central Water Power Commission and it is proposed to develop gradually this National Network of Waterways. It is proposed in the Master Plan (as noted below) to link up all the important rivers, one with the other, wherever possible.

* **Master Plan** :—

(1) Linking the Ganga River system with the Narmada (a west flowing river) to provide a continuous waterway from the west coast to the east coast.

(2) Connecting the Narmada with the Godavari river system.

(3) Linking the Taptti with the Godavari System via the Wardha, a tributary of the Godavari.

(4) Linking the Ganga river with the Mahanadi via the Sone and Rihand.

(5) Inland Coastal waterways connecting Calcutta with Madras, via Cuttack along the east coast, and then round the Cape to the west coast connecting the existing canals and backwaters of Kerala and Mysore.

Extensive surveys and investigations are necessary to find out the feasibility of all these schemes.

2. Necessity for Inland Navigation :—Inland Navigation is generally recommended wherever it is possible, because

(a) The transport of goods by water is generally cheaper than transport by rail.

(b) The load transported by water is greater than in railways (if there are proper arrangements).

(c) Railways and Motors have failed to meet the requirements of transportation adequately.

(d) Inland transportation by water can be used as an alternative means of transportation, in times of emergency for the defence and security of the country.

* Taken from *Bhagirath* for April-57.

Recently there are proposals to improve inland waterways and the navigable canals on a large scale, and the Central Water and Power Commission is at it. Inland Navigation is also given a place in the First Five Year Plan of India, and it is proposed to explore Inland Navigation in every river valley project.

3. Waterways:—For Inland Navigation, first, there should be a proper waterway, and this may be a river, or a big distributary or a big canal. The canal may be either a canal constructed for navigation only, or a canal constructed for both navigation and irrigation.

4. Requisites of a navigable canal:—For navigation, a canal constructed for the purpose should be as follows:—

(a) It should have sufficient cross sectional area and depth to accommodate the boats used. The minimum width of the navigable canal should be more than 20 ft. To meet this, a wide cross sectional area should be maintained from the upstream reach to the downstream reach of the navigable canal.

(b) There should be ample supply of water for the boats to move easily. Boats when fully loaded usually sink down to about 3 ft. and as such a depth of 4 ft. or more is an advantage as the haulage is much lighter with at least 1 ft. of water below the boat.

(c) The velocity of water flowing in the canal should not hinder the navigation or the movement of the boats, because towing the boat against the flow of water is difficult, especially when the velocity of the current is more than 2 ft.

5. Navigation on Irrigation canals:—Whether to combine navigation with irrigation or not, has become a problem with expert engineers to solve. The extra cost of construction and maintenance involved with regard to the design of the works and the regulation of flow, affect the efficiency of the canal.

In deltaic regions, the profit from navigation increases inevitably where stone is not available and cart traffic is only possible on metalled roads in rainy seasons. The yield from the fields can also be easily transported by navigation in such places. It is desirable to have navigation combined with irrigation only in large deltaic regions.

Every large irrigation scheme will come short of its utility if it is not combined with navigation. No one will object to having cheap transportation by navigation. A combined navigation and irrigation canal will also be a financial success, (but a purely irrigation canal will be more so.)

6. Requirements of combined navigation and irrigation canals :—The requirements of a purely navigation canal already given above, will obstruct the purposes of irrigation completely, if they are provided as on the irrigation canal. For an irrigation canal the requirements are

(a) The cross sectional area of the canal should gradually decrease from head to the tail reaches of the canal, as the area to be irrigated decreases.

(b) The canal should have sufficient velocity, to avoid silting and scouring.

(c) The canal should have a surface fall and a current.

Therefore canals constructed for both navigation and irrigation, should satisfy the following conditions :—

(a) The canal should have a large cross sectional area, and at the downstream reach, there should be more water than required for irrigation.

(b) The velocity of water in the canal should be at the lowest non-silting velocity suitable for the required depth (as the cost of navigation increases with the velocity).

In some cases, this velocity is as required for irrigation, but high for navigation, and the towing of the boats against the current will be difficult.

(c) During periods of maximum demand, the canals should be given a surface slope to pass the water required for irrigation, and even when the demands of irrigation are small, the canal must be run with a sufficient depth throughout as required for navigation.

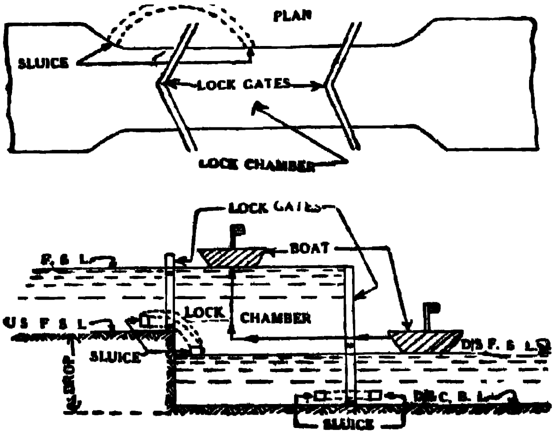
7. Works required for a combined navigation and irrigation canal :—The navigation works required for a canal constructed for both irrigation and navigation are

(A) Canal Lock :—A lock called '*Canal lock*' should be provided where there is a drop in the canal. This is a chamber

built of heavy flooring or masonry between the reaches of (two) canals at different bed levels (the difference in the reaches is called the '*Lift*' *of the lock*).

Watertight gates are provided for the lock chamber, and boats can enter or leave the chamber from either end, when the gates are opened. These gates are called *lock-gates* and are provided in pairs, pivoted on a vertical axis in a recess in each longitudinal wall of the chamber. The outer vertical edges of the gate make an angle pointing upstream, since the lock entrance width is smaller than the width of the two gates together. The masonry abutments receive the thrusts of water on the gates along the horizontal beams. (See Fig. 291)

CANAL LOCK



Section

Fig. 291.

One end of the lock-chamber meets the upstream side of the weir and the other end meets the downstream side of the weir. Sluices were provided at both ends of the lock chamber. When the gates are closed, the sluices allow the chamber to be filled from the upstream end of the canal or to be emptied to the downstream end of the canal. In both cases, the sluice shutters are operated by rack and pinion method. Rack and pinion method is preferred

to screw gear method, because it can be operated quickly and saves time. (For filling or emptying, it generally takes only 3 minutes).

The dimensions of the largest size of boats to be accommodated should be considered, while fixing the minimum dimension of the lock chamber. The chambers are so designed as to accommodate two or more crafts at the same time when the traffic is heavy. These are of two classes according to the requirements of traffic (i) 150 ft. × 20 ft. and (ii) 105 ft. × 15 ft.

The sills of the lock chamber at its upstream and downstream ends are at the bed levels of the upstream and the downstream side of the weir. When the gates are closed, the lower horizontal edges of the gates rest on these sills. On the downstream side of the lock-weir a road bridge is provided in a line with the bridge over the lock-weir.

At the upstream side of the lock chamber, two pairs of gates, facing in opposite directions, are provided, to remove the excess of water due to the rise of water level above the full supply level, at the discharging tail locks of a river due to flood and also to raise the lock walls and bank connections above the high flood level.

A double lock is usually provided at a point, when the fall at a lock is more than 10 ft. to 12 ft. at that point along the alignment of a canal. This lock consists of two chambers, to divide the total fall into two steps, thus reducing the head on each set of gates which are three in number with sluices between the upper and the lower chamber. Hence they can be operated easily.

(B) Lock Weir :—For navigation, a combination of regulators and drops called Lock weirs are necessary. The depths of water above the lock required for navigation and that necessary in the reach below required for irrigation, are both simultaneously regulated by these lock weirs. These are specially required at drops, when irrigation and navigation are combined.

A lock should be provided for the boats between the lock weir and the approach. The arrangement of the lock chamber and the lock weir is as follows :—

(a) The lock weir can be provided across the line of the canal and the lock chamber on a diversion (See Fig. 292).

LAYOUT OF LOCK WEIR AND LOCK CHAMBER

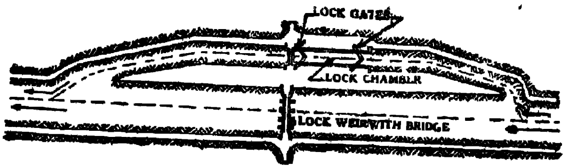


Fig. 292

This is usually adopted when the weir stream is very large as compared to the lock chamber. This arrangement gives room to silting of the navigation canals, especially at the room between the point where the canal leaves it and the point where it rejoins it. Navigation will be obstructed by this silting, and hence it has to be frequently cleared.

(b) The weir can be provided on a deversion and the lock chamber across the line of the canal (See Fig. 293). This arrangement is preferred to the first.

(c) The lock weir and the lock chamber can both be provided adjacent to each other, one work being provided across the canal and a row of fender piles fixed between the approaches of the lock weir and the lock chamber (See Fig. 294). In this arrangement silting is avoided, by a different approach to the lock chamber. Sufficient security can also be obtained by suitably locating the fender piles and the mooring posts. This arrangement will be the best.

(C) Fender Piles :—Fender piles are usually provided at surplus sluices or off-takes for the canal, to overcome the difficulty of movement of the boats. The piles are of wood or R. C. C. and fixed generally 8 ft. to 10 ft. centre to centre, when there are no off-takes.

Fender piles are used as piers in the construction of light tow path bridges between the approach channel and the off-take (See Fig. 295).

LAYOUT OF LOCK WEIR AND LOCK CHAMBER

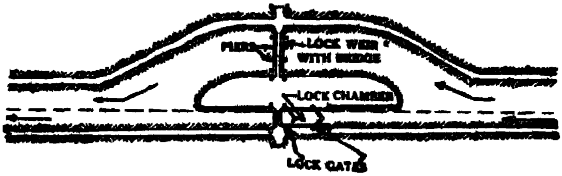


Fig. 293

LAYOUT OE LOCK WEIR AND LOCK CHAMBER

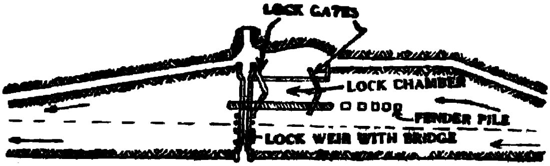


Fig. 294

SKETCH SHOWING FENDER PILES USED AS
PIERS FOR A TOW PATH

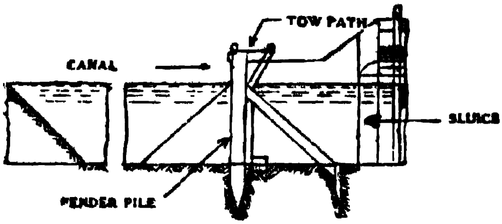


Fig. 295

(D) Tow Paths:—To make the movement of the boat easy against the current, tow paths are provided throughout the length of the canal.

Tow paths of different sizes are provided according to different conditions as noted below: (a) A tow path is provided along the edge of the berms, where they are above the full supply level. (b) A tow path of 2 ft. in width is provided 1 ft. above the full supply level along the inner edge of the berm where the above berms are wide and below the full supply level.

(c) A tow path of 4 to 5 ft. in width is provided ½ to 1 ft. above the full supply level through the side span of the bridge across the canal.

(E) Bridges: Bridges provided on navigation canals should have enough space above the high flood level, to accommodate freely the boats. The size of the boat used should be taken into consideration in the actual construction of the bridge. (Arched bridges will not satisfy all the requirements of the headway, and so Girder bridges are preferred).

8. Advantages and Disadvantages of Combined Navigation and Irrigation. The advantages of combining navigation with irrigation are

(a) The cheap transportation.

(b) The direct route obtained for the traffic.

The disadvantages are

(a) Expenditure due to the construction of works necessary and their maintenance.

(b) If the velocity of the water is restricted to meet the requirements of navigation, silting and scouring may occur.

(c) Impossibility of competing with railways (in cost of transportation).

(b) Danger of placing the lock weir, which is subjected to heavy stress due to the changes in hydrostatic pressures on its structure.

(e) Likelihood of the supply of water for irrigation being affected if the safety of the lock is to be guarded.

Canals constructed for purely irrigation purposes should not be used for navigation on large scales (only small boats which do not obstruct the purposes of irrigation can be used).

9. Waterways in Russia:—*Russia completes the Volga-Don canal.* Russia's great dream of linking its two important water-roads, the Volga and the Don rivers, became a reality (according to U. S. S. R. Information Bulletin).

As a result 18,000 miles of navigable river in the Volga and the north-western basins and more than 7800 miles in the Don and

Dnieper basins, have become a single network of deep waterways. This system links all the seas of the European part of the U. S. S. R.

The Volga-Don Project will eventually supply water to about 6,790,000 acres of arid land in the Rostov and Stalingrad regions. It will also provide cheap electric power for industry and agriculture in the irrigated regions.

Special Design:—In the canal route area, the Don waters are 144 ft. above the Volga level. This made pumping stations necessary, to feed water from the Don to the canal and its locks. By entering the canal from the Volga end, ships should climb the 288 ft. of the Volga slope, by a stairway of nine locks. After crossing a flat section of the divide, the ships descend 144 ft. to the level of the Don, by means of four locks.

The great importance of the Volga-Don canal is its communication with the Baltic, White, Caspian, Azov and Black seas. Certainly the new canal will strengthen the intricate Soviet canal system.

This Chapter will not be complete, without a Note on T. V. A., and hence the following small note is added.

10. Tennessee Valley Authority—T. V. A. *(A Note)*: The original settlers or colonists from Britain denuded all the forest area to get immediate good profits and amass money, and exported all the timber available.

With the object of getting minerals for export, the ground was excavated and made unfit for cultivation. When the mining operations were in swing, the fumes from smelting caused the killing of all vegetation nearby and as a result of this, the rains caused the erosion of the soil.

When the wood was all exhausted and lumbering had to be stopped, the colonists took to agriculture. The slope of the earth was ploughed and hence there was heavy loss of top soil and water. The colonists abandoned this area. Thus by shifting cultivation from place to place, the whole area was laid bare. The basic productivity of the soil being stripped over a vast area, it hurried along to sterile waste in the sea. (The soil which was thick even up to three feet or more in places was now laid bare as rocks unfit for cultivation).

Rich valleys were first occupied by the early settlers. Later on the text batch of settlers got only higher lands and these being not fertile, overfarming was adopted to get more crops for export and hence forests were stripped bare.

To improve the eroded portion or the gullied area, aforestation was adopted and community forests were started. Terraces were formed in the land, to protect the crops.

Note: It is said that about 3000 million tons of soil are washed away into the sea from U. S. A. every year.

The row-cropping method allows the top soil to be washed away. The over-grazing of the land leads to the same result.

I It had its own independent organisational structure.

II Its own special methods of survey.

III Its system of research adjusted to the needs of the consumer.

IV Its achievement (due to planning) in the all-round development and general welfare of the people in the area.

1. **Organisation.** There was decentralisation of administration.

The town-planning expert, research expert, and the educational expert were all independent of each other.

The secret of success of the organisation was, that all the problems met with were solved on the spot.

The organisation had control over the interstate navigation, in

1 Improvement of the waterways,
2 Interest in flood control,
3 — do — Irrigation,
4 Improving the electric power,

Navigation :—The rivers in the area (of the scheme) are made navigable for a long distance (even about 650 miles), and thus the inland waterway system is much improved; and on this account, the transportation has become cheap.

The navigation system consists of (a) navigation locks and (b) dredger channels.

Function of navigation authorities—

1 They prepare navigation charts.

2 They make comparative studies of the railway and water freight charges.

3 They prepare traffic estimates.

4 They identify the commerce that might be transported economically.

II Survey : For land survey, the following details are employed.

1 Aerial photography.

2 Detailed soil-survey.

3 Types of soil are graded into 5 classes according to their productivity.

4 Detailed site survey. — These are divided under 4 heads.

(a) Sites of historic interest.

(b) Sites of natural beauty.

(c) Sites of interesting geological formation.

(d) Sites of interesting plant or animal life.

III Research —

The organisation have made the result of research work available to the public, and invited the co-operation of firms and even private individuals in the matter.

Scientific Farming :—Improved methods of agriculture protect the land from wastage. Demonstration farms have been introduced here and there for the cultivators to copy. The cultivators have been induced to use fertilisers to their lands (which in return give them 350 per cent increase in the output of crop).

N. B. :—To prevent lands from being damaged, it is advised that steeper lands should be left as a permanent feature, and on lands with gentle slope, sodd crops should be sown, and in level country, row crops should be grown.

IV Achievement. *General development* :—Electric Power was developed by the electric-grid system, and this permitted supply of power to big factories like Alcova—the largest aluminium plant in the world. Power is used by all factories, big and small. The whole industry is stimulated, as cheap electric power is made available even for heavy industry, and for making use of the ore

deposits in the locality. Small industries also spring up in many places.

Industrial Progress :—All industries were improved especially preparation of war materials (because it was war period) and fertilisers, etc. The coal and ice merchants opposed the scheme because it produced cheap power which was detrimental to their profits.

Welfare : All the workers were housed very well, even during the construction period. Near the dam site (even during progress of work) recreation areas were built. In the colony for the labour and the staff, schools, library and shopping centres were estabiished; and even in offices, club rooms were maintained. Other facilities like a post office, a telephone exchange, a modern well-equipped restaurant, cafetaria for workers and stores for food stuffs were all provided for. Kitchens were also provided here and there, to save labour to the workmen. Thus a decent and pleasant town was constructed for the workmen.

Constructions of Buildings :—Houses.

The houses were of demountable type. These could be easily dis-assembled, moved to a new site, and assembled there easily. They were cheaper, only the distance to be conveyed should not be more than 250 to 300 miles. If more than this distance, another type called the panel type of buildings was used.

The houses were scattered all about the area with wooded and gentle slopes given. For the construction of houses, simple materials and gay colours were used to produce a cheerful effect. For example, for a railway station, a clear-cut plan with an open shelter is given and bright colours are used.

Recreation :—When reservoirs were formed by the construction of dams, the fishing industry increased (even to about 75 times). To provide shelter and food to these fish, a sort of aquatic vegetation was grown. These fisheries not only provided food to the people, but also gave them recreation sailing, canoeing, motor-boating, etc.

Tourist Traffic :—The preservation of Natural beauty in the region is of the utmost importance in attracting tourists. For the convenience of the tourists, the buildings and the roads were

designed, especially to suit their tastes; for example, scenic highways, look out buildings, patrol stations, and camps appropriate to valley conditions were all constructed. Also picnic shelters in the parks, and simple unpainted timber construction for shelter were provided to match with nature. The cabins for tourists were built in groups. Facilities for boating and bathing for the tourists, and wading pools for children, were also provided for.

Special visitors' buildings with rest houses, kiosks, and overlooking terraces have been built. Circular buildings, high enough to see the fine mountains and the lake views, in all directions, have been constructed. In the visitors' buildings refreshments are arranged to be served on terraces under gay umbrellas. Reception rooms that are built are made to overlook the reservoir.

The roads in the locality are constructed not only for the visitors to enjoy the scenery all round, but the road itself has become a landscape asset. For the roads, lamps are provided without any glare. Flush light boxes with patent lenses are also provided.

The scenery (unspoiled by man's haphazard developments) includes the most primeval forests. One could see the rocky peaks, waterfalls buried in the forest mountain laurels, and natural bridges all round.

APPENDIX

* **Hydrograph**: Hydrograph is a graph showing the flow of stream at a particular point in its course. This graph is obtained by plotting time along the X-axis and the discharge along the Y-axis.

It should be noted that the flow rises suddenly after a rainfall, and after attaining the peak point, decreases slowly and gets back to the normal base flow.

Unit Hydrograph :—Unit Hydrograph is a hydrograph representing 1 in. of runoff for an isolated rainfall of some unit (dimension) duration and specific areal distribution.

The rates of runoff from consecutive units of rainfall-excess having the same areal distribution will be proportional to the unit hydrograph and the ordinates obtained by multiplying the unit hydrograph by rainfall-excess amount of unit duration. i. e. rainfall in excess of *n* inches within the unit duration will give a runoff represented by a hydrograph having the ordinate *n* times that of the unit hydrograph.

Unit hydrographs resulting from rainfall-excess quantities uniformly distributed over a drainage basin may be used to compute rates of runoff that would be under average rainfall conditions. On the other hand, unit hydrographs reflecting the regimen of runoff from precipitation of somewhat higher intensity in the lower basin may be used in estimating the critical rates of discharge. Minor variations in the rainfall distribution may be eliminated by having valley storages, but major variations in the distribution will be reflected in the runoff hydrographs.

Unit Rainfall Duration :—This is the duration of runoff producing rainfall, or rainfall-excess, that results in a unit hydrograph. The term *lag*, as used herein, is the length of time from the midpoint of the unit rainfall duration to the peak of the unit hydrograph.

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* Notes taken from Engineering for Dam-by Creager, Justin & Hinds.

The unit-rainfall duration selected for a unit hydrograph should not exceed the period over which the designed storm rainfall is assumed to be approximately uniform in intensity in various portions of the drainage area under study. For drainage areas larger than approximately 100 sq. miles, a 6 hr. unit-rainfall duration is suitable and convenient for most purposes of study; and for very large areas even up to 1 day unit-rainfall duration may be adopted. The longer the intervals, the more will be the variation in the areal distribution of the rainfall. Hence a longer interval may be selected to eliminate the effects of major variations in the distribution, but the result becomes approximate in value.

Three methods are usually adopted for developing the unit hydrographs. They are

(1) By analysis of rainfall-runoff records for isolated "Unit Storms"

(2) By analysis of rainfall-runoff records for major storms

(3) By computation of Synthetic Unit Hydrograph.

(1) **Unit Hydrograph from Isolated Unit Storm:** This is the most direct method of deriving a unit hydrograph involving the analysis of runoff records resulting from an isolated unit storm, that produces reasonable uniform rainfall-excess rates for a period approximately equal to the desired unit-rainfall duration. The usual procedure adopted is as follows:

(a) Prepare a map showing an outline of the basin, the location of stream gauging stations and precipitation station in and near the basin.

(b) Construct a net work of *Thiessen* polygons covering the basin under study.

(c) Determine the date of occurrence of the periods of rainfall over the basin, that appear to have been reasonably well isolated from other periods, and inspect the precipitation records (as obtained from Weather Bureau Climatological Data).

(d) Determine approximately the volume of runoff from each of the rainfall periods as considered above, by referring to stream flow records. From this, choose an isolated storm of unit duration (fairly uniformly distributed over the catchment area) so that the conditions will simulate those of intense rainfall.

(e) Prepare mass rainfall curves for precipitation stations and near the basin for the period selected.

(f) Plot the discharge hydrograph of the river (i) some days prior to and (ii) some days after the rainfall.

(g) Draw the *Depletion Curve* (This is tangential to the lowest point of the hydrograph).

(h) Modify the oberved hydrograph as required to exclude runoff from extraneous rainfall, and estimate the base flow. Find out the volume of surface runoff (This is the area between the actual hydrograph and the line of base flow), by subtracting the base flow from the total hydrograph of runoff resulting from the unit storm. Convert this volume into inches of runoff over the whole catchment area.

(i) Plot this data in the form of *hydrographs*, to indicate the areal distribution and intensity characteristics of the runoff producing rainfall that may have had an important effect on the regimen of runoff.

(j) Divide the ordinates of the hydrographs of surface runoff resulting from the Unit Storm by the volume under it, expressed in inches of runoff from the drainage area, to obtain the unit hydrograph.

(k) Measure the ordinates of the depletion curve and deduct them from the corresponding ordinates of the observed hydrograph, to give the difference in the flow due to the storm.

(l) If the duration of rainfall-excess during the storm differs appreciably from the unit-duration adopted for general use, the computed unit hydrograph may be adjusted to the desired unit-duration.

N. B..—The unit duration should always be less than 24 hours (12 or even 6 hours is preferred).

Time of Concentration is the time between the time of starting of the intense rainfall and the time of the start of the peak flow.

Unit Hydrograph is generally got from a compound hydrograph; because it may not be always possible to get a convenient record for a storm of unit duration to draw a unit hydrograph.

The features of this method are :—

(i) Variation of intensities during a storm affect the peak discharge and the rate of runoff.

(ii) The rate of rainfall cannot be taken from the average.

(iii) The observed hydrograph takes into account the storage, infiltration and surface detention.

(iv) Ordinates of the unit hydrograph are proportional to the total volume of the surface runoff from the unit time rainfall (depth of rainfall is not taken into account).

(v) The rate which produces the surface runoff in a unit time is constant.

(vi) The distribution of the runoff is constant.

N. B.:—If instead of 24 hours, a duration of 12 hours is taken, the unit hydrograph will change, also the shape of the unit hydrograph depends upon the duration of the storm.

Unit hydrograph cannot be applied to runoff from snow or ice.

(2) **Unit Hydrograph from Major Flood Records:** A unit hydrograph derived from a unit storm, or trial graph developed by synthetic methods, is first applied to the computed rainfall-excess values to obtain a hypothetical hydrograph for comparison with the observed hydrograph. Modifications in the lag and the shape of the unit hodrograph are made as required, following the S-curve procedure to obtain a reasonably close agreement between the actual and computed hydrographs.

The results obtained generally may be considered more reliable for deriving the designed floods than those obtained by anaiysis of minor runoff hydrographs resulting from unit storms, although results of each method should be checked against each other.

The differences between unit hydrographs derived from minor and major flood records are

(a) In the case of minor floods, there is approximately uniform areal distribution of rainfall; but in the case of major floods the rainfall intensity and the accumulated amount vary over the area. If the volume of rainfall-excess during the major storm was proportionately heavier in the lower part of basin, or near the principal stream channels, the concentration of the runoff would

be higher than that represented by the unit hydrograph derived from the minor flood rises.

(b) Unit hydrographs derived from major flood hydrographs for small streams would have higher peak discharge ordinates than those derived from minor floods.

(3) **Synthetic Unit Hydrograph:** Synthetic unit hydrographs are required either as a substitute for derivations from hydrologic records or as a means of correlating and supplementing the observed data, in the case of majority of hydrological studies. There are several methods of computing the synthetic unit hydrograph each method being developed to serve some special purpose. It may not constitute the most suitable procedure for certain other uses.

Selection of Unit Hydrographs for Designed Flood Computations: It is usually assumed that a unit hydrograph, applicable to the most intense periods of rainfall during a designed storm would have a higher peak dischage ordinate, and would represent a higher concentration of runoff, than might be indicated by unit hydrographs derived from minor floods. Uncertainities regarding the proper unit hydrograph values for use in estimating the designed flood runoff are substantially reduced, if adequate and reliable hydrologic records are available for floods that resulted from rainfall intensities and areal distributions reasonably comparable to those to be expected during the designed storm. In order that the use of unit hydrographs derived from available hydrologic records in computing the designed flood discharges may assure conservative results, they have to be modified to represent higher rates of runoff.

It is usually desirable to determine the amount of increase in the maximum reservoir level that would result from various differences in the concentration of runoff from the desined storm before final decisions regarding the selection of unit hydrographs are attempted when developing a spillway designed flood for a reservoir project.

Reservoir Inflow Unit Hydrograph:—The regimen of flood runoff may be materially altered by the synchronization of high rates of runoff originating above the head of the reservoir with the maximum rates from areas contributing laterally to the reservoir,

by the formation of a *long reservoir* in a natural drainage basin. The runoff from the upper portion of a basin, under natural river conditions, is retarded by valley storage, and normal frictional resistance as it passes through the reservoir reach, resulting in a velocity as that for an open channel. The inflow near the upper end of the reservoir moves through the pool largely by a process of translation, with long wave velocities subject to momentum control, after a deep reservoir has been formed by the construction of a dam.

Changes in the synchronization of runoff from various portions of a drainage basin may be so as to produce rates of inflow into a full reservoir that are substantially higher than would occur at the dam site under natural river conditions, although in some reservoirs the difference may be negligible.

GLOSSARY

Adasli is a sugar cane crop getting watersupply beyond its usual or sanctioned time (of getting water). The Adsali cane is planted in July and harvested in November of next year.

Accretion of levels—This is the converse of Retrogression of levels. It is a rise in the level of the bed of a stream at any particular site.

Avulsion—If a river—specially in alluvial soil—changes its course entirely and adopts a new course damaging property, etc., it is called Avulsion. The river generally breaks through the narrow neck of the horse-shoe bend of the river, or through banks (sometimes).

Baffle wall is a wall constructed to dissipate the energy of falling water from a weir. The wall is built at a distance (say 5 times the height of drop) from the toe of the fall.

Banjar is an unculturable land.

Creep—When a structure is built on a permeable soil, water finds its way out either under the structure or round it. This is called *creep*.

Dentated sill—This is built at the end of an apron, below a fall. It is a notched sill intended to check the force of falling water and thus prevent erosion or scour.

Chak is the area irrigated from a sluice.

Intermittent and periodical canal —This a canal taken from a stream, and used only temporarily when there is no rain. It is generally small and used by individual ryots.

Kalar is a soil consisting of salts unfit for irrigation.

Khodva is the crop got from the first cutting of sugar cane, or it is the first rattoon. The second cutting is called *Nidva* and the third rattoon *Rodva*.

Kist is land assessment paid by instalments.

Leaching is the washing out of salts from the upper reaches of a land during floods.

Loop bund is a subsidiary bund constructed a little distanc behind the main bank of a river. It is built as an additiona precaution against danger of breach of the main bank.

Minor is a branch of the distributary of a channel. The name given to a minor is usually that of the village to whose lands water is supplied.

Mori is a pipe-outlet; it is also called *Colaba*.

Nappe is the sheet of water flowing over a weir or dam. The nappe has an upper and a lower surface

Needle—This is a device used for controlling water from a dam or weir. This may be of wood, metal or a combination of both. It is generally inserted vertically and hence the name *needle*.

Open discharge is the mean discharge of a canal for the number of days it was in flow that is—

Pre-seasonal watering—Before the beginning of irrigation season, water is taken for certain land either to prepare the land or to water the tender crops. This is called pre-seasonal watering (Generally, a single water rate is charged for this).

Pick-up-weir—This is a weir or anicut built lower down the main reservoir, to pick up or use the water at the place, more conveniently for irrigation.

The water so picked up, may be that from the main reservoir which is situated in a hill tract higher up, where there is not much available land for irrigation nor labour. The pick up weir being situated in the plain lower down, there is both land and labour.

The water may be mostly waste water, already used for irrigation once, and simply going to waste. This is picked up at a convenient place and used again for irrigation.

Rajbaha or *Rajakalve* (Raya kalva) is the name given to the main distributary channel.

Staunching wall—This is a wall constructed at the junction of earthwork and masonry to prevent percolation or leakage. The wall increases the length to be traversed by the creeping water, and it increases the frictional resistance to the flow, as it is constructed at right angles to the wing wall.